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b. Pages 27 – 65: "Asphalt Underlayment Railway Trackbeds: Designs, Applications, and Long-Term Performance Evaluations" by Jerry G. Rose.
c. Pages 66 – 86: "A Laboratory Test Method for Measuring Realistic Trackbed Pressures at the Tie/Ballast Interface" by Jerry G. Rose, David B. Clarke, Qunglie Liu, and Travis J. Watts.
d. Pages 87 – 112: "In-Track Railway Track Tie/Ballast Interfacial Pressure Measurements Using Granular Material Pressure Cells" by Jerry G. Rose, David B. Clarke, Qunglie Liu and Travis J. Watts.
e. Pages 113 – 137: "Comparisons of Railway In-Track Tie/Ballast Interfacial Impact Pressure Measurements with Wheel/Rail Surface Impact Load Detector" by Jerry G. Rose, Travis J. Watts, and Ethan J. Russell.
f. Pages: 138 – 169: "Modeling Crosstie-Ballast Load Distribution in a Railroad Trackbed via Finite Element Analysis" by Brent D. Thompson, David B. Clarke, and Jerry G. Rose.

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NURail Project

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Asphalt Underlayment Highway/Railway At-Grade Crossings: Designs, Applications, and Long-Term Performance Evaluations

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TECHNICAL SUMMARY

Title

Asphalt Underlayment Highway/Railway At-Grade Crossings:
Designs, Applications, and Long-Term Performances Evaluations

Introduction

Rehabilitation and/or renewal of highway/railway at-grade crossings frequently accounts for major track maintenance expenses for railroad companies and governmental agencies in the United States. The jointly used area represents a significantly expensive unit cost of the highway and railway line. Within this area, the crossing surface and trackbed replace the highway pavement structure.

An at-grade crossing is designed to fulfill its primary purpose of providing a clear and smooth path for the safe and comfortable passage of highway vehicles across railroad tracks and ideally will maintain a smooth surface and stable trackbed for a long period of time. At-grade crossings with long service lives greatly reduce the likelihood of costly and frequent disruptions that result from recurrent surface repairs.

Approach and Methodology

Historically, crossings have been expected to deteriorate at a faster rate and require reconstruction at more frequent intervals than the adjacent pavement and open-track. Structurally, railways and highways are typically designed very differently, but must co-exist within the common area of the crossing. When the roughness and deterioration of the crossing adversely affects the safety and reasonable traffic operations across the crossing, the crossing must be removed and replaced at high cost and inconvenience to the traveling public and railroad operations. If the crossing is replaced using similar materials and techniques, this can assure the occurrence of a similar series of events.

This report describes contractual arrangements, guidelines, standards, and design practices used by representative public transportation agencies and railroad companies to rehabilitate a grade crossing using asphalt underlayment technology. Detailed documentations of long-term performance evaluations of asphalt underlayment crossings are described and discussed for these selected state DOTs, public commuter/transit lines, and freight railroads involved with the development and adoption of this technology for various operating conditions throughout the United States. Evaluations are conducted for crossings consisting of the eight most common types of crossing surfaces.

Findings

In recent years, numerous public transportation agencies and railroad companies have shown increased interest in adopting improved trackbed crossing designs to provide enhanced structural capability, thereby lengthening service lives and improving the performance of at-grade highway/railway crossings. Several public transportation agencies and railroad companies have developed and now specify guidelines and standards for the proper design and construction of highway/railway at-grade crossings that incorporate asphalt underlayments. Contractual arrangements and design practices used by ten representative public transportation agencies and railroad companies, described herein, attest to the successful application of this technology.

Long-term performances of hundreds of crossings incorporating asphalt underlayment indicate overwhelming justification for its use. No crossing deterioration as a function of trackbed pumping, settlement or premature deterioration of the crossing surface- indicative of inadequate trackbed support- has been detected for a wide variety of operating conditions. The only failure mechanism exhibited for a limited number of crossings is deterioration of the crossing material which can be readily replaced with minimum expense and interruption to highway and railway traffic.

Conclusions

Adding a layer of asphalt when constructing a new trackbed or renewing an existing trackbed will conceivably increase the cost of the crossing project compared to using an all-granular trackbed. However, for crossings that routinely exhibit short service lives due to unfavorable site conditions and poor performance, the additional cost for asphalt underlayment is minimal relative to the total cost of renewal. Typically, these crossings require the removal and replacement of the existing support and track materials with a premium surface applied. Often times, the added cost for the asphalt layer is less than 5 percent of the total renewal cost. If the crossing protection equipment is also replaced, the added percentage increase in cost will be even less. Though in most cases the cost of asphalt underlayment is minimal, it is important to note that the benefit-cost ratio varies from project to project. For crossings requiring frequent renewals, the added cost for the layer of asphalt will be more economically justifiable. Conversely, crossings that rarely exhibit structural deficiencies and maintain long service lives may not warrant the additional cost.

The time required to renew a crossing with a layer of asphalt varies considerably. This will largely depend on the project size and scope, as well as the pre-project planning and administration. There have been cases where two-lane highways have been completely re-opened to rail traffic within four hours and highway traffic within 8 to 12 hours. In other cases, crossing renewals that have involved additional appurtenant activities have required several days. The specifics of the train traffic and highway traffic at a given crossing will undoubtedly affect the planned crossing outage.

Recommendations

The findings and conclusions contained in this report are largely qualitative. The evaluations consider the longevity of acceptable performances for many types of crossings containing many different types of crossing surfaces. The scope spans many areas of the country and analyzes crossings on both heavy rail tonnage/high highway traffic freight lines and lower tonnage/high highway traffic commuter/transit lines. Although the conclusions appear somewhat repetitious for the ten agencies, the absence of available and proven quantitative evaluation measures makes it necessary to monitor a large and widely variable sample of crossings to accurately assess the performance of asphalt underlayment over long periods of time.

At present, there is not an established quantitative measure for gauging the performance of highway/railway at-grade crossings. A simple, direct measure of the relative rideability of crossings reflective of the crossings' smoothness/roughness and its adverse effect on vehicular driver/passenger comfort and safety is desired.

Publications

Rose, J.G. and B.R. Malloy. "Highway-Railway At-Grade Crossing Designs Containing Asphalt Underlayments". PROCEEDINGS of the 2017 Annual Conference of the American Railway Engineering and Maintenance-of-Way Association. September, 2017, 15 pages.

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SECTION 1 - INTRODUCTION

Two very dissimilar modes of transportation jointly utilize highway/railway at-grade crossings. An adequately designed and maintained crossing provides a clear and smooth path for the safe and comfortable passage of highway vehicles across railroad tracks. The adequacy of the support structure within the crossing to accommodate the combined highway and railway loadings is a key aspect in assuring long service lives and desired performance levels for crossings. In recent years, numerous public transportation agencies and railroad companies have shown increased interest in adopting improved trackbed crossing designs to provide enhanced structural capability, thereby lengthening lives and improving performance measures of at-grade highway/railway crossings. An option during new construction or renewal of an existing crossing is to place a layer of asphalt pavement (termed "asphalt underlayment") below the ballast, similar to that used as highway paving material, to achieve an improved structural and waterproofing support layer. Numerous public transportation agencies and railroad companies now consider, based on an engineering evaluation of the particular site conditions, the use of asphalt underlayments when rehabilitating highway/railway at-grade crossings.

Rehabilitation and/or renewal of highway/railway at-grade crossings frequently accounts for major track maintenance expenses for the railroad companies and governmental agencies in the United States. The jointly used area represents a significantly expensive unit cost of the highway and railway line. Within this area, the crossing surface and trackbed replace the highway pavement structure.

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Historically, crossings have been expected to deteriorate at a faster rate and require reconstruction at more frequent intervals than the adjacent pavement and open-track. Structurally, railways and highways are typically designed very differently, but must co-exist within the common area of the crossing. When the roughness and deterioration of the crossing adversely affects the safety and reasonable traffic operations across the crossing, the crossing must be removed and replaced at high cost and inconvenience to the traveling public and railroad operations. If the crossing is replaced using similar materials and techniques, this can assure the occurrence of a similar series of events.

SECTION 2 - BACKGROUND

2.1 Typical All-Granular Trackbed Support

Historically, the most common track (sub-structural) support for highway/railway crossings is the typical trackbed consisting of unbound all-granular materials as depicted in Figure 2.1a. When the inherent lack of support for the highway vehicles in the track crossing area results in excessive deflections of the crossing, the excessive deflections, combined with the lessening of the support strength due to the high moisture contents of the support materials, can ultimately result in permanent settlement of the crossing. This can adversely affect the vertical profiles of the highway and railroad in the immediate crossing area. An ideal trackbed design might include a high-quality substructure (or base) below the trackbed to provide similar load carrying, confining, and waterproofing qualities to the common crossing area – as typically exists in the abutting pavement sections.

2.2 Typical Asphalt Underlayment Trackbed Support

The use of a layer of hot mix asphalt within the track substructure – in lieu of, or in addition to, conventional all-granular subballast - is becoming widely utilized to provide ideal properties to the crossing (1,2). During the past thirty years, hundreds of crossings have been rehabilitated or initially constructed using this procedure. The basic process involves removing the old crossing surface and track panel followed by excavating the underlayment mixture of ballast, subballast, and subgrade to the required depth. These are replaced with compacted layers of granular subballast (optional), hot mix asphalt (termed "asphalt underlayment"), and ballast forming the structural support for the new track panel, and a new crossing surface, as depicted in Figures 2.1b and 2.1c.

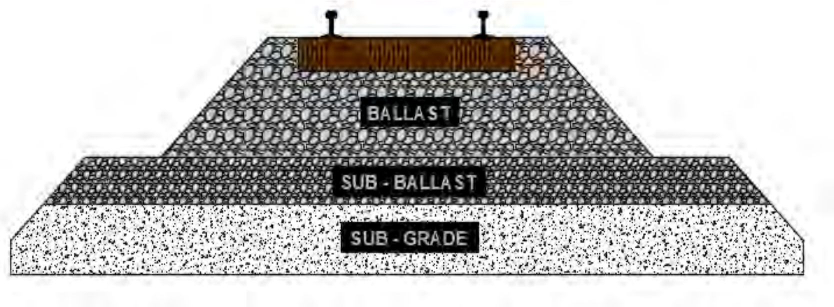
The addition of the layer of asphalt provides the ideal sub-structural support system for a highway/railway crossing; these being (3):

- Produces adequate strength to resist the combined highway and railway loadings thus minimizing stresses on the underlying subgrade,
- Minimizes vertical deflections and permanent deformations of the crossings due to highway and railway loadings so that the wear and deterioration of the crossing components will be minimized, and
- Serves to waterproof the underlying subgrade so that its load carrying capability will not be sacrificed even when placed on marginal quality subgrades.

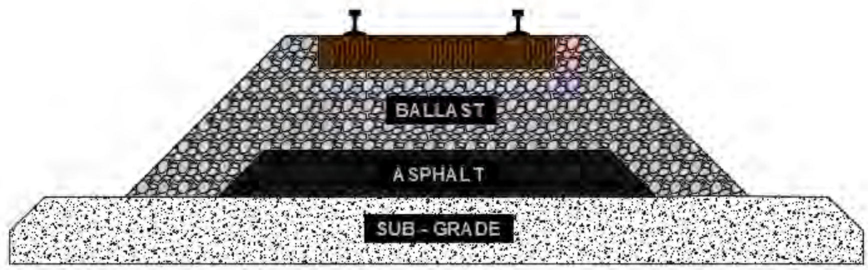
Numerous long-term tests and performance evaluations attesting to the superior performance of asphalt trackbed crossings have been conducted on heavy trafficked highway/railway crossings in Kentucky. The tests and corresponding results are described elsewhere (2,3,4,5).

2.3 Typical Asphalt Underlayment Trackbed Designs for Crossings

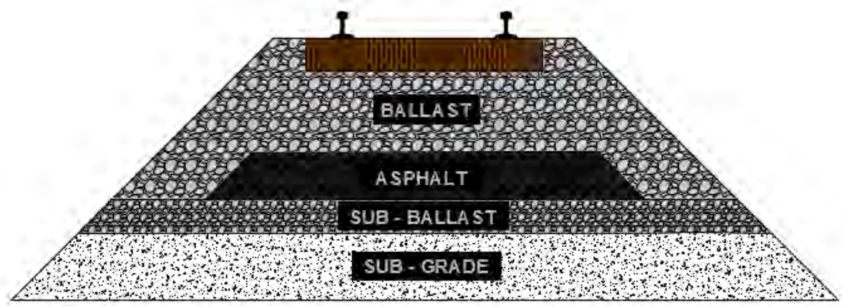
Typical dimensions for the asphalt underlayment layer are 12 ft (3.7 m) wide and 6 in. (150 mm) thick. For very poor trackbed support conditions and high impact areas, an 8 to 12 in. (200 to 300 mm) thickness of asphalt is commonly used. Thickness of the overlying ballast ranges from 8 to 12 in. (200 to 300 mm). Thickness of a granular subballast layer below the asphalt, if utilized, is usually 4 to 8 in. (100 to 200 mm) thick. The length of the asphalt layer will normally extend for a specified distance beyond the immediate crossing area. This distance is based on prevailing conditions at the specific site and the time available to perform the work. A distance of 10 to 20 ft (3 to 6 m) or more is desirable.



a) Typical All-Granular Trackbed



b) Asphalt Underlayment Trackbed



c) Asphalt Combination Trackbed

Figure 2.1. Typical All-Granular Trackbed (a) and Two Variations of Asphalt Trackbeds (b & c).

The asphalt mixture specification is normally the prevailing dense-graded highway base mix in the area having a maximum aggregate size of ¾ to 1½ in. (25 to 38 mm). The asphalt binder content can be increased by 0.5% above the optimum value for highway applications resulting in a low to medium modulus (plastic) mix, having design air voids of 1 to 3%. This mix is easier to densify to less than 5% in- place air voids and therefore facilitates adequate strength and an impermeable mat. Rutting of the plastic mix is not a concern in the trackbed since the pressures are applied through the ballast over a wide area. Bleeding and flushing are also of little concern since the wheels do not come in direct contact with the asphalt layer and the temperature extremes are minimized in the insulated trackbed environment (6,7,8,9,10,11).

SECTION 3 - EXTENT OF UTILIZATION OF ASPHALT UNDERLAYMENT CROSSINGS

The use of asphalt underlayment has become increasingly significant in this country at select track sites and unique situations to provide increased structural support and waterproofing during maintenance and replacement of special trackworks - specifically for turnouts, crossovers, rail crossings, and wheel impact load detectors. Additional applications, similar in scope, include paving tunnel approaches and inverts and bridge approaches for various distances. A limited number of reasonably long sections of open-track, especially on capacity improvement projects, have had asphalt underlayment utilized at selected sites for specific reasons to justify the increased initial costs (12,13).

The large Class I railroad companies in the U.S. have been selectively using asphalt underlayment for new at-grade crossings and the rehabilitation of existing at-grade crossings based on engineering analyses of the benefits and logistics for the particular crossing site. Many regional and shortline railroad companies are involved as well. A select number of public agencies - including state public utility commissions, state DOTs, and urban commuter/transit lines - are participating with railroad companies in specifying and funding applications of this technology. Application is becoming a standard practice for selected railroads and public agencies for crossing renewals where the prevailing conditions have adversely affected the performance of conventional all-granular supported crossings. Several public agencies specify the use of asphalt trackbeds for all crossing renewals on which public funds are used (14,15,16,17,18).

Following are descriptions of contractual arrangements and design practices used by ten representative public transportation agencies and railroad companies that have taken proactive approaches to adopting this technology. The programs vary considerably among railroads and agencies; the following discussions are merely samples. Detailed documentations of long-term performance evaluations are described and discussed for these state public utility commissions, state DOTs, public commuter/transit lines, and freight railroads for widely varying operating conditions throughout the United States.

3.1 Portland & Western and WES/TriMet

The Portland & Western Railroad (P&W), a Genesee & Wyoming subsidiary, began using asphalt underlayment in 2006 during the renewal and upgrading of highway/railway crossings on its northwestern Oregon lines. Historically, maintaining serviceable highway/railway crossings presented challenges for this 520 mile (837 km) regional railroad. Frequent crossing renewals were common in this high annual rainfall climate on lines traversing less than desirable native subgrade soils. Muddy and settled crossings were common. However, since the adoption of asphalt underlayment and insuring that adequate drainage is incorporated with the impervious asphalt mat, the crossings have performed adequately with no mud pumping or track settlement. No extra track surfacing, crossing surface maintenance, or renewals have been required. For discussion, the P&W applications of asphalt underlayment are separated into three distinctly different categories of applications.

3.1.a Regular Street and Highway Crossings

During the past eleven years the P&W has renewed 4 to 6 typical highway/railway crossings per year using asphalt underlayment. The total number in service is around fifty with additional installations planned for 2017 and succeeding years. A portion of these were partially funded by the Oregon Department of Transportation, particularly on high-volume highway traffic crossings in urban areas. The typical track support section is composed of 6 in. (150 mm) of granular subballast, 6 in. (150 mm) of asphalt underlayment, and 12 in. (300 mm) of ballast.

3.1.b Long-Distance Street-Running Continuous Crossings

Of particular significance to the P&W is the predominance of long-distance middle-of-the-street running trackage in several cities. This was common to the predecessor Oregon Electric Line that operated frequent interurban passenger trains in the Portland suburbs. Crossings ranging from 2000 to 3500 ft (600 to 1100 m) long are common. These have typically required frequent and costly track maintenance activities to maintain smooth surfaces for both train and highway vehicle passage. This has included frequent surface replacements, roadbed injection, pavement milling, etc. Providing adequate drainage throughout these long distances to minimize settlement and deterioration and provide adequate trackbed support for the jointly used crossings has been difficult to achieve using conventional all-granular support structures.

Renewing the crossing surfaces and support structures for the poorly performing middle-of-street crossings began in 2009 in Independence with a 2000 ft (600 m) long Main Street crossing installed over four years using asphalt/rubber seal surfaces with concrete at selected cross streets.. This was followed with a 3,500 ft (1,100 m) long Holly Street concrete crossing in Junction City, depicted in Figure 3.1, also installed over four years. The P&W trackage has 2.9 miles (4.7 km) of middle-of-street-running crossings, scattered over six cities.

Performance of the two crossings to date has been very satisfactory. During 2017 P&W plans similar projects in two cites – 2,800 ft (850 m) long Front Street in Salem and 1,400 ft (425 m) long 4th Street in Harrisburg. These will be multi-year projects following the specifications and procedures used for the initial two long-distance street crossings.

3.1.c WES/TriMet Commuter Line

A short section of the P&W trackage is also utilized by the Westside Express Service (WES) Commuter Line, an extension of the extensive TriMet rail passenger system in the Portland area. Sixteen commuter trains operate weekdays in each direction during peak AM and PM periods. The line serves freight trains during the mid-portion of weekdays, nights and weekends.

The line was extensively upgraded in 2007/2008 for the initiation of commuter traffic and improved and increased freight traffic operations. Various improvements were made to the track, including installing new/larger rail and concrete ties. Twelve of the 17 public street crossings on the line were deemed in need of renewal and subsequently fitted with asphalt underlayment during the upgrade. An additional WES crossing was renewed with asphalt underlayment in 2010. This crossing at Durham Avenue is shown in Figure 3.1.

Figure 3.1. The 3,500 ft (1,100 m) long Holly Street Middle-Of-Street Crossing in Junction City (left) containing Asphalt Underlayment. Durham Avenue (right), one of 13 Crossings on the WES Line containing Asphalt Underlayment.

The in-service WES crossings are performing adequately with no settlement or deterioration. Wood ties and Pandrol clips were used in the immediate crossing areas rather than concrete ties. Asphalt underlayment is planned when the remaining four at-grade crossings require replacement and renewal.

3.2 West Virginia Department of Transportation

The WVDOT began utilizing asphalt underlayments during the rehabilitation of crossings in 2000. Its use soon accelerated. For several years, when WVDOT funds are used for crossing rehabilitation projects on state highways, the use of asphalt underlayment has been considered a standard practice. Since 2000, an average of seven to eight crossings has been underlain with asphalt per year. It is estimated that over 145 crossings have asphalt underlayment, the oldest having been in service for 16 years. Most of these renewals have been on heavy tonnage, high traffic crossings and a large percentage have routinely exhibited deterioration soon after renewing using standard granular trackbed support materials.

Normal practice is to use a high-type surface material, commonly concrete precast panels, and improved support and drainage, achieved with a 6 in. (150 mm) thick asphalt underlayment. The asphalt layer is normally placed 12 ft (3.6 m) wide and extends 10 to 20 ft (3.0 to 6.1 m) beyond the crossing surface based on an engineering evaluation of the site. This practice qualifies as a betterment program to upgrade crossings for improved performance and increased service life. This is considered a benefit to the traveling public. WVDOT funds the crossing surface material differential (premium material vs. railroad standard material), asphalt underlayment, traffic control, drainage pipe (if needed), and tie differential for longer ties. The railroad company contributes the costs of labor and equipment.

Since the program began, no crossings have failed due to lack of substructure support or excessive settlement; all have remained smooth and serviceable. A nominal number have required surface replacement after ten or so years due solely to the deterioration of the surface materials particularly on heavy traffic crossings.

WVDOT standards dictate that the minimum crossing service life (assumes the crossing remains reasonably smooth) should be a minimum of ten years. This long life expectancy also enables the railroad company to "skip" crossings at least once for 5-year maintenance planning, only needing to routinely adjust the geometry of the open track. Numerous crossings with asphalt underlayment have exceeded the useful life expectancy of ten years, a tremendous benefit to the traveling public since the crossings are seldom closed for renewal. Previously the service lives for many of these crossings, particularly in the heavy coal-haul routes, were as short as two years. Figure 3.2 contains recent views of two WVDOT long-life crossings that have not had any maintenance since they were installed, although the pavement approaches are showing some deterioration.

Figure 3.2. WV Route 2 at Ashton (left) placed in 2001 and 5th Avenue W (US 60) in Huntington (right), placed in 2000. These are exemplary of WVDOT Asphalt Underlayment Crossings exceeding the minimum Service Life of Ten Years. The only Deterioration is slight raveling of the Asphalt Approaches.

3.3 Caltrain

This 55 mile (88 km) long regional rail link along the San Francisco Peninsula began using asphalt underlayment in 1999 for the rehabilitation/renewal of at-grade street crossings, pedestrian crossings, and station platforms. During the intervening 18 years, 21 multi-track street crossings, involving 43 track underlayments, and 8 station platforms, involving 19 track underlayments throughout the lengths, have been rehabilitated/renewed using this technology. This mixed traffic line carries caries frequent commuter trains, over 500 per week, and a limited number of UP local freight trains during the evening hours.

Construction details are contained in Caltrain's Standard Drawings and Practices. An engineering evaluation is performed to determine unique conditions at each site that might influence the specific design. After removing the old crossing surface and roadbed materials, considerable attention is given to preparing the roadbed for the trackbed. This involves drainage considerations and thorough compaction of the existing or modified roadbed, ensuring a well-compacted 6 in. (150 mm) minimum thickness of compacted subgrade/roadbed.

The materials for the asphalt underlayment, ballast, and crossing surface must conform to the provisions of Caltrain's Standard Specifications. The underlayment mix is designated dense-graded Type A Mix with ¾ in. (19 mm) maximum size aggregate gradation. It is placed as a layer 8 in. (200 mm) thick, with either a machine or blade, and thoroughly compacted. The underlayment layer is placed 12 ft (3.7 m) wide and extends a minimum of 10 ft (3 m) along the track beyond the ends of the crossing surface. A layer of ballast, 9 in. (225 mm) thick, is placed above the asphalt. Precast concrete panels are considered standard crossing surfaces.

The crossings have all exhibited satisfactory performance since installation. No subgrade support issues have been observed and the crossings remain smooth and serviceable, some being in service for as long as 18 years. Based on demonstrated performance and engineering evaluations, the use of asphalt underlayment is considered the standard design for railway/highway at-grade crossings, pedestrian crossings, and station platforms on Caltrain's mainline.

3.4 Metrolink

This large commuter rail system in the Los Angeles area of Southern California presently consists of seven lines totaling 388 miles (624 km) in extent and continues to expand with additional lines. Metrolink began using asphalt underlayment in 2007 for at-grade railway/street crossings. Since that time, numerous crossings have been underlain with asphalt. It is considered standard practice for all newly constructed and rehabilitated crossings. The design is similar to that specified by Caltrain, except that only 6 in. (150 mm) thickness of asphalt underlayment is used. The designs are contained in Metrolink's Standard Book of Specifications.

The performances of the crossings have been satisfactory. Minimal settlement has been observed and the crossings have remained smooth and serviceable, requiring no maintenance. Recent views of typical streets on Caltrain and Metrolink underlain with asphalt are shown in Figure 3.4.

3.5 Illinois Commerce Commission and Department of Transportation

The Illinois General Assembly established the Grade Crossing Protection Fund (GCPF) in 1955. The program is administered by the Illinois Commerce Commission (ICC) and the funds are distributed by the Illinois Department of Transportation (IDOT). The ICC has primary administrative authority over approximately 7000 local roads and street public crossings and IDOT has primary authority over approximately 700 state roads and highway public crossings on the state road network.

Utilizing a portion of the GCPF for rehabilitation/renewal of highway-rail grade crossing surfaces began in 2009 with state fiscal year 2010 (July 1, 2009), as a component of the ICC

Figure 3.4. Caltrain's San Mateo 1st Street Crossing and Station Platform (left) placed in 2,000 using Hot-Mix Asphalt Mixture and Metrolink's Osborne Street Crossing (right) in the Sun Valley Area placed in 2013 using a Cold-Mix, Polymerized Emulsion Asphalt Mixture. These Crossings have exhibited Excellent Performance, Typical of all Caltrain and Metrolink Street Crossings Underlain with Asphalt.

crossing safety improvement program. GCPF assistance is granted on a per-request basis. Railroads may only apply for assistance on surface renewal projects at grade crossings that are located on local roads and streets. A formal letter of request, following specific guidelines established by the ICC, must be submitted. The GCPF is used to reimburse railroads for all materials, including contract labor (asphalt paving, traffic control, etc.). The railroads cover all labor costs to install new crossing surfaces.

To qualify for assistance, selected grade crossings must meet several requirements set forth by the ICC; primarily the highways' annual average daily traffic (AADT) volumes and whether a crossing is along a designated truck route. The ICC specifies apportionment of the crossing surface rehabilitation component of the GCPF based on railroad class; Class I railroads receive 75%, Regional railroads receive 5%, and Short-line railroads receive 20% of the allocated funds.

The primary funding source for IDOT grade crossing surface renewal projects is the IDOT Central Office appropriations distributed to each district on a yearly basis. Federal Safety Funds (Section 130) can include a crossing surface, but only if an appropriate signal upgrade is also part of the project. Additional local and state funds may also be used. As is the case with projects funded by the GCPF, the Section 130 fund covers only the cost of materials. Additional costs, including labor and miscellaneous costs are absorbed by the railroad.

Beginning in FY2010 the railroads began to selectively seek reimbursement for installing asphalt underlayments. Asphalt underlayment is required for all crossing surface renewals administered by the ICC on designated truck routes regardless of AADT, and for all crossings on roads/streets with AADT > 5,000 vehicles per day. Since 2010, 32 crossing surfaces have been rehabilitated with asphalt underlayment on the ICC network of local roads and a similar number rehabilitated with asphalt underlayment on the IDOT highway network.

The asphalt layer is normally specified as 6 in. (150 mm) thick, 12 ft (3.6 m) wide, and extends a minimum of 25 ft. (7.6 m) beyond ends of the crossing. The crossings containing asphalt underlayment, located mainly on high volume highway and truck routes, have performed to ICC and IDOT expectations. Additional crossing surfaces are scheduled for utilization of asphalt underlayments in 2017 on the ICC and IDOT renewal programs; the on-going practice continues to expand in scope statewide.

3.6 Indiana Shortline Railroads

Two reasonably-sized shortline railroads, both with primary trackage in the state of Indiana, have been proactive in recent years with the use of asphalt underlayment for the rehabilitation, renewal, and upgrading of at-grade crossings. It has been necessary for both of these shortlines to upgrade the quality and class of the lines to accommodate increased rail traffic volumes. The Indiana Department of Transportation (INDOT), using state funds and contributions from local highway authorities, has funded a portion of the crossing projects. However, public funds are used only under the condition that significant upgrades to the highway and street aspects of the crossings are included in the project.

3.6.a Louisville & Indiana (L&I) Railroad

The L&I Railroad is a 106 mile (171 km) line providing a direct north-south route through the south-central portion of the state from Indianapolis to Louisville, KY. The line is currently being significantly upgraded to carry increased rail traffic in conjunction with CSX Transportation.

This railroad began installing asphalt underlayments in the late 1990s during the rehabilitation and upgrading of numerous at-grade crossings that had degraded due to deferred maintenance of the crossing surfaces and trackbed. By 2010 L&I had installed asphalt underlayments at 30 or more crossings, about two-thirds with partial funding from public funds. These crossings represent many of the heaviest highway traffic crossings on the line. Figure 4 depicts a typical crossing.

During the present programmed track upgrades, particularly the renewal of the rail with larger size new rail, many of the crossings - which had been renewed in recent years with new track panels and crossing surface and improved trackbed support with asphalt underlayments - did not require upgrades. The performances of these particular crossings were deemed adequate for several more years of service. The rail-exchange team merely “skipped” the crossings, thus providing minimal disruption to the traveling public and savings for the railroad company. One of these crossings is depicted in Figure 3.6a. Some of the shorter crossings in the rural areas were temporarily removed recently during the rail renewal and track upgrade.

The crossings containing asphalt underlayments have provided satisfactory performance all along the line, providing smooth, level crossings for normal operation of the railroad and acceptable comfort and safety for the motoring public.

Figure 3.6a. Charlestown-New Albany Road Crossing in Jeffersonville (left) installed in 2003; State Route 256 Crossing in Austin (right) installed in 2007, Recent Rail Upgrade is shown on Left Bottom, (inset bottom right shows Sept. 2015 Weld), the 9-Year Old Existing Crossing was Unaffected.

3.6.b Indiana (INRD) Railroad

The INRD Railroad is a 250 mile (400 km) line providing an east-west route through the southwest portion of the state from Indianapolis to southern Illinois. The line has been significantly upgraded in recent years to carry increased coal volumes and intermodal traffic. This railroad began installing asphalt underlayments about eight years ago during the rehabilitation and upgrading of numerous at-grade crossings in anticipation of the increased rail traffic volumes. During the past eight years INRD has installed upwards of 25 or more crossings in the two states, several of these have involved partial funding from Indiana and Illinois public agencies. Most of the crossings upgraded with asphalt underlayment and premium crossing materials have been on high volume highway crossings and in urban areas.

Of particular significance is the enhanced quality of crossings in the city of Bloomington. Four high-volume highway crossings, three within close proximity to the University of Indiana, were renewed and upgraded with premium crossing surfaces and asphalt underlayments during 2011 and 2012.

The crossings containing asphalt underlayments have provided acceptable performances in both of the states of Indiana and Illinois. All crossings have remained smooth and serviceable assuring typical operations for the railroad and desired comfort and safety for the motoring public. INRD continues to selectively use asphalt underlayments during the rehabilitation and renewal of highway/railway crossings having concrete panel surfaces installed.

<u>*3.7 Denver's RTD FasTracks – Eagle P3 Project*</u>

FasTracks is Denver's RTD (Regional Transportation District) voter-approved transit expansion program – the largest in the nation – transforming transportation through the Denver metro area.

This program, approved in 2004, augments the earlier completed light rail passenger lines serving the Denver Metro Area.

The Eagle P3 Project was approved in 2010. It includes the construction of three commuter lines as part of RTD's commuter rail line system. The project is being delivered and operated under a concession agreement that RTD entered into with a "Concessionaire" known as Denver Transit Partners (DTP); the team is composed of several large companies. The Eagle P3 Project concession agreement requires DTP to design, build, finance, operate, and maintain three lines – A line, G. line, and B line. The A & B lines opened in 2016; the G line is anticipated to open in 2017.

Of specific interest to this discussion was the decision by the DTP's design team to specify the use of asphalt underlayment designs for trackbed support within the at-grade railway-highway/street crossings for the three new lines. The fact that the long-term maintenance agreement that DTP must adhere to for the next thirty years dictated that premium designs, materials, and construction techniques should be used to minimize subsequent maintenance expenditures.

There are 52 at-grade highway/street crossings along the three routes, 10 of these crossings are located in flood plains requiring direct fixation crossing design containing ballastless tub concrete crossings positioned on lean concrete base. The remaining 42 crossings contain typical ballast and granular subballast including asphalt underlayments.

The specific asphalt underlayment trackbed design layers consist of 5 in. (125 mm) asphalt underlayment, placed 15 ft. (4.6 m) wide extending 10 ft. (3.0 m) beyond each end of the crossing. The asphalt layer is topped with 12 in. (300 mm) ballast for the typical ballasted track.

Maintenance requirements and serviceability measures will be closely monitored during succeeding years for these three lines to determine the effects of the specific designs on the costs and warranty implications of the executed contractual obligations of the concessionaire. One of these studies will evaluate whether the additional cost of installing asphalt underlayments under the highway/street crossings and turnouts will be recovered due to expected lowered maintenance costs and improved service metrics.

3.8 Iowa Department of Transportation

The Iowa Department of Transportation (IowaDOT) initially instituted an at-grade crossing management program in 1973 with the introduction of the Grade Crossing Surface Repair Fund (GCSRF). In 1999 all primary crossings were taken from the database and ranked by condition criteria and funded under the Primary Grade Crossing program using $1 million dollars from the Primary Road Fund.

A ranking system is used to determine priorities for funding crossing surface renewals using Primary Grade Crossing Funds. The system is based on prioritizing nineteen engineering factors unique to the specific crossing. Crossing life expectancy for crossings renewed with conventional all-granular support was often only two years for crossings that historically exhibited structural problems and associated pumping/settlement. During the first two years of the Primary Program, 30 to 35 crossings were funded to address the degraded crossings quickly.

Figure 3.7. RTD Crossing under Construction in 2013 (left) and finished view in 2017 of Miller Street double track crossing on the G Line (right) with BNSF Line at far left.

Beginning in late 2000 the IowaDOT began using asphalt underlayment during the renewal of selected crossings, particularly those exhibiting short service lives. Since that time the backlog of eligible projects has significantly decreased and by 2010 the yearly renewals were reduced to less than five as the service lives of the crossings underlain with asphalt were extended significantly requiring less frequent renewals.

It is estimated that since 2000, 90 to 100 crossings on the IowaDOT primary system have been underlain with asphalt. It is considered standard practice when IowaDOT funds are utilized to renew/upgrade crossings.

On crossings where asphalt underlayment has been used, no crossing failures have occurred due to a lack of structural support or excessive settlement when specified IowaDOT practices are followed. The service lives for the asphalt underlayment crossings have increased significantly. A few precast concrete panels have cracked under particular impact loadings and required replacement, but no settlement issues were involved. Additionally, railroad production track maintenance work can normally skip the crossings for at least one or two maintenance cycles since only minor settlement is observed and only normal weathering of the exposed crossing materials is evident. Based on sixteen years of observing the improved performance and longer service lives of at-grade crossings underlain with asphalt, numerous benefits have been gained by public agencies and railroad companies:

3.9 Kentucky Transportation Cabinet

The Kentucky Transportation Cabinet (KYTC) began evaluating asphalt underlayment in combination with economical timber/asphalt crossing surfaces during the renewal of five L&N/SBD Railroad (later CSX) heavy traffic at-grade crossings in Central Kentucky during the early/mid 1980s. Before renewal, these crossings were frequently in deplorable conditions, exhibiting a combination of mud pumping, excessive settlement, and surface deterioration - all of which contributed to unacceptable rideability conditions and led to frequent maintenance and replacement. During the succeeding thirty years these crossings have performed very well, only requiring renewal of the economical crossing surfaces at 12 to 15-year intervals as a result of deterioration of the surface materials due to long-term weathering and exposure. Figure 3.9 depicts the current condition of a CSX crossing renewed with asphalt underlayment in 1984.

The application of asphalt underlayments by KYTC during the renewal/rehabilitation of crossings began in earnest about 2000 and during the succeeding twelve years, 68 crossings were underlain with asphalt. The predominance of these crossings was initially in Eastern Kentucky on heavy-haul coal highways and CSX coal lines. Later crossings sites included NS and Paducah & Louisville and CSX lines throughout the state. The crossings that were selected had historically required frequent maintenance and renewal to maintain acceptable serviceability and ride quality. Figure 3.9 shows the condition of a CSX crossing in western Kentucky ten years after renewed with asphalt underlayment.

The long-term performances of the 73 at-grade crossings in Kentucky containing asphalt underlayments within the track substructure, for a variety of typical crossing surface materials, were documented in a Kentucky Transportation Center 2014 report (16). The findings from the study indicated that the performance of crossings rehabilitated/renewed with asphalt underlayment had been excellent, with no crossing failures due to excessive settlement, ballast fouling, or mud pumping. The services lives had been significantly improved, as the crossings had remained smooth and serviceable for longer periods than the preceding crossings that used conventional all-granular renewal techniques. A small percentage of the crossings had experienced deterioration of the crossing surface materials; this largely due to inherent aging of the surface materials.

Further evaluations indicated that economical crossing surfaces, such as rubber seal/asphalt and timber/asphalt, can be expected to perform as well as their more expensive premium counterparts provided the crossing support is adequately designed, the crossing is properly installed, and efficient drainage is achieved.

Figure 3.9. Pike Street Crossing (left) in Cynthiana, renewed with Asphalt Underlayment in 1984, Surface replaced twice during the 32 intervening years. US 60 Crossing (right) at Stanley, ten years after renewal with Asphalt Underlayment, Still in perfect condition in 2012.

KYTC's early experimentation and documented experiences with asphalt underlayment for at-grade crossing rehabilitation/renewals has served as a precursor for numerous other railroads, state DOTs, and governmental agencies.

SECTION 4 - CONCLUDING REMARKS

In recent years, numerous public transportation agencies and railroad companies have shown increased interest in adopting improved trackbed crossing designs to provide enhanced structural capability, thereby lengthening service lives and improving the performance of at-grade highway/railway crossings. Several public transportation agencies and railroad companies have developed and now specify guidelines and standards for the proper design and construction of highway/railway at-grade crossings that incorporate asphalt underlayments. Contractual arrangements and design practices used by ten representative public transportation agencies and railroad companies are described herein.

At present, there is not an established quantitative measure for gauging the performance of highway/railway at-grade crossings. A simple, direct measure of the relative rideability of crossings reflective of the crossings' smoothness/roughness and its adverse effect on vehicular driver/passenger comfort and safety is desired.

The findings and conclusions contained in this report are largely qualitative. The evaluations consider the longevity of acceptable performances for many types of crossings containing many different types of crossing surfaces. The scope spans many areas of the country and analyzes crossings on both heavy rail tonnage/high highway traffic freight lines and lower tonnage/high highway traffic commuter/transit lines. Although the conclusions appear somewhat repetitious for the ten agencies, the absence of available and proven quantitative evaluation measures makes it necessary to monitor a large and widely variable sample of crossings to accurately assess the performance of asphalt underlayment over long periods of time.

Adding a layer of asphalt when constructing a new trackbed or renewing an existing trackbed will conceivably increase the cost of the crossing project compared to using an all-granular trackbed. However, for crossings that routinely exhibit short service lives due to unfavorable site conditions and poor performance, the additional cost for asphalt underlayment is minimal relative to the total cost of renewal. Typically, these crossings require the removal and replacement of the existing support and track materials with a premium surface applied. Often times, the added cost for the asphalt layer is less than 5 percent of the total renewal cost. If the crossing protection equipment is also replaced, the added percentage increase in cost will be even less. Though in most cases the cost of asphalt underlayment is minimal, it is important to note that the benefit-cost ratio varies from project to project. For crossings requiring frequent renewals, the added cost for the layer of asphalt will be more economically justifiable. Conversely, crossings that rarely exhibit structural deficiencies and maintain long service lives may not warrant the additional cost.

The time required to renew a crossing with a layer of asphalt varies considerably. This will largely depend on the project size and scope, as well as the pre-project planning and administration. There have been cases where two-lane highways have been completely re-opened to rail traffic within four hours and highway traffic within 8 to 12 hours. In other cases, crossing renewals that have involved additional appurtenant activities have required several days. The specifics of the train traffic and highway traffic at a given crossing will undoubtedly affect the planned crossing outage.

Long-term performances of hundreds of crossings incorporating asphalt underlayment indicate overwhelming justification for its use. No crossing deterioration as a function of trackbed pumping, settlement or premature deterioration of the crossing surface- indicative of inadequate trackbed support- has been detected for a wide variety of operating conditions. The only failure mechanism exhibited for a limited number of crossings is deterioration of the crossing material which can be readily replaced with minimum expense and interruption to highway and railway traffic.

SECTION 5 - ACKNOWLEDGMENTS

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The initial interest and funding for developing this technology was provided by CSX Transportation and the Kentucky Transportation Cabinet during the 1990s and early 2000s. Former CSX Transportation and Engineering Officer Gerald L. Nichols is specifically commended for his initial interest and continued support at the University of Kentucky culminating in the development and subsequent application of the highway/railway at-grade crossing technology described herein.

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Asphalt Underlayment Railway Trackbeds: Designs, Applications, and Long-Term Performance Evaluations

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TECHNICAL SUMMARY

Title

Asphalt Underlayment Railway Trackbeds: Designs, Applications, and Long-Term Performance Evaluations

Introduction

This compendium of practices report documents the justification, design and implementation of technical advancements for a wide variety of asphalt trackbed applications. Documented are over forty specific projects on four Class I railroads, two regional/shortline railroads, plus significant applications on three public commuter/transit rail lines These include special trackworks (crossovers, rail crossings, turnouts), wheel impact load detectors (WILD), bridge approaches, tunnel floors and portals, open-track, and middle-of-street trackage applications.

Approach and Methodology

Typical and specific design and application practices were evaluated. The longevity of these applications varied from recent to over thirty years of in-track service. Emphasis was placed on evaluations of performance metrics affecting subsequent train operations, track maintenance requirements and costs, and overall maintenance of track geometric parameters. The performance evaluations are largely based on anecdotal and qualitative observations and historical evaluations augmented with limited quantitative track geometry adherence data and subsequent track maintenance cost data

The projects were designed and installed using highly technical standards of engineering practice relating to the developed technology for using a layer of asphalt, termed "underlayment", within the track structure to achieve specified high standards for trackbed performance. The coverage reported herein is not all-inclusive, but represents typical, pertinent, and unique applications of which the author is familiar.

Findings

In the early 1980s, several U.S. railway companies saw the impending need for higher-quality trackbed materials to provide higher performance and longer-lasting track and support structures. The railway companies worked with the asphalt paving industry to assess the applicability for using a layer of hot-mix asphalt within the track structure to replace a portion of the conventional granular material. The initial emphasis was primarily on heavy-tonnage freight railroads, employing asphalt for trackbed maintenance applications and solving instability problems in existing trackbeds. The majority of these trackbed applications involved selectively installing a layer of asphalt during the rehabilitation of turnouts, railroad crossings, bridge approaches, defect detectors, hump tracks, tunnel floors and approaches, highway crossings, and loading facilities where conventional trackbed designs and support structures had not performed satisfactorily. These asphalt maintenance installations currently number in the thousands. Based on its proven performance as a maintenance solution, asphalt is also selectively considered as an option on new mainline track installations in the U.S.

During the intervening years since the early1980s, the specified use of a layer of hot-mix asphalt underlayment in the track structure (trackbed) -- as a replacement or partial replacement for granular subballast below the ballast -- has grown substantially on U.S. freight railways and public commuter/transit rail lines. Typically this practice is specified at specific sites on existing trackage where conventional all-granular trackbed designs have not performed satisfactorily and/or where trackbed enhancement features potentially afforded by the asphalt layer were deemed particularly desirable.

The asphalt layer is normally used in combination with traditional granular layers to achieve various component configurations. This practice augments or replaces a portion of the traditional granular support layers and is considered to be a premium trackbed design. The primary documented benefits are to provide additional support to improve load distributing capabilities of the trackbed layered components, decrease load-induced subgrade pressures, increase confinement for the ballast, improve and control drainage, maintain consistently low moisture contents in the subgrade, insure maintenance of specified track geometric properties for heavy tonnage freight lines and high-speed passenger lines, and decrease subsequent expenditures for trackbed maintenance and component replacement costs.

Conclusions

During the past several decades designs incorporating a layer of asphalt (or bituminous) paving material as a portion of the railway track support structure have steadily increased until it is often considered as a common or standard practice.

The observed conditions and evaluated performances of all described projects indicate that asphalt trackbeds are meeting or exceeding anticipated longevity and performance expectations.

Recommendations

The United States railway industry continues to emphasize the importance of developing innovative trackbed design technologies for both heavy tonnage freight lines and commuter/transit passenger lines. The purposes are to achieve high levels of track geometric standards for safe and efficient train operations while minimizing long-term track maintenance costs and extending track component service lives.

Based largely on its proven performance it is considered a standard engineering design practice for specified types of applications on a variety of freight railroads and commuter/transit rail lines in the United States. This technology also has demonstrated applications for the construction of numerous new high-speed passenger lines in Europe and Asia.

Publications

Rose, J.G. "Asphalt Railway Trackbed Designs, Applications, and Long-Term Performances". PROCEEDINGS of the 2017 Annual Conference of the American Railway Engineering and Maintenance-of-Way Association. September, 2017, 24 pages.

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SECTION 1 - INTRODUCTION

During the intervening years since the early1980s, the specified use of a layer of hot-mix asphalt underlayment in the track structure (trackbed) -- as a replacement or partial replacement for granular subballast below the ballast -- has grown substantially on US freight railways and public commuter/transit rail lines. Typically this practice is specified at specific sites on existing trackage where conventional all-granular trackbed designs have not performed satisfactorily and/or where trackbed enhancement features potentially afforded by the asphalt layer were deemed particularly desirable.

This compendium of practices report documents the justification, design and implementation for a wide variety of applications of this technology involving over forty specific projects on four Class I railroads, two regional/shortline railroads, plus significant applications on three public commuter/transit rail lines These include special trackworks (crossovers, rail crossings, turnouts), wheel impact load detectors (WILD), bridge approaches, tunnel floors and portals, open-track, and middle-of-street trackage applications.

Typical and specific design and application practices are described. The longevity of these applications varies from recent to over thirty years of in-track service. Emphasis is placed on evaluations of performance metrics affecting subsequent train operations, track maintenance requirements and costs, and overall maintenance of track geometric parameters. The performance evaluations are largely based on anecdotal and qualitative observations and historical evaluations augmented with limited quantitative track geometry adherence data and subsequent track maintenance cost data. The observed conditions and evaluated performances of all described projects indicate they are meeting or exceeding anticipated longevity and performance expectations.

SECTION 2 - BACKGROUND

In the early 1980s, several U.S. railroad companies saw the impending need for higher-quality trackbed materials to provide higher performance and longer-lasting track and support structures (1,2,3). They worked with the asphalt paving industry to assess the applicability for using a layer of hot-mix asphalt within the track structure to replace a portion of the conventional granular material. The initial emphasis was primarily on heavy-tonnage freight railroads, employing asphalt for trackbed maintenance applications and solving instability problems in existing trackbeds. The majority of these trackbed applications involved selectively installing a layer of asphalt during the rehabilitation of turnouts, railroad crossings, bridge approaches, defect detectors, hump tracks, tunnel floors and approaches, highway crossings, and loading facilities where conventional trackbed designs and support structures had not performed satisfactorily (4,5). These asphalt maintenance installations currently number in the thousands. Based on its proven performance as a maintenance solution, asphalt is also selectively considered as an option on new mainline track installations in the U.S.

Similar cooperative efforts in several European and Asian countries have been primarily directed at high-speed passenger lines. This involves the construction of new segments or complete rail lines using asphalt (frequently termed *bituminous*) trackbeds. Their engineering and construction approaches are relevant in the U.S. today because the next large expansion of our rail system may be high-speed passenger rail. The contents of this paper are limited to U.S. freight railroads and passenger commuter/transit lines. Its applications on European and Asian countries are described in previous papers (6,7).

SECTION 3 - ASPHALT TRACKBED DESIGN PARAMETERS

Typical asphalt trackbed design parameters have changed significantly since the initial trials during the early 1980s. Design specifications have become more specific and stringent relative to using basic engineering principles and practices for selecting materials quality, layer thicknesses, and in-track installation best practices. Accepted practices from long-standing highway and airfield technologies have influenced the technical aspects specified in the railway recommended practices.

3.1 Trackbed Cross-Sectional Configurations

Three basic types of asphalt trackbeds are being utilized, as depicted in the cross-sections contained in Figure 3.1. Two of them incorporate the traditional ballast layer as a portion of the support. The so-called "*Asphalt Underlayment*" trackbed is similar to the classic all-granular trackbed; the sole difference being the substitution of the asphalt layer for the granular subballast layer. The asphalt is placed on a select subgrade.

The "*Asphalt Combination*" trackbed includes both an asphalt layer and a granular subballast layer placed on either new subgrade or existing trackbed. The thickness of the asphalt layer can be somewhat less than that used for underlayment since a relatively thick specified granular subballast layer is placed below. This is normally the type of asphalt trackbed that is in existing trackage since the in-place supporting material is typically a mixture of degraded ballast, soil, and other fine materials that have accumulated over the years. This is mainly granular material with a grading similar to subballast. It is quite variable in composition and quality, but typically has higher strength than native soil. The native subgrade soil is normally imbedded several more inches below the degraded subballast layer.

The "*Ballastless Asphalt Combination*" trackbed consists of rail/tie track, or slab track, placed directly on a relatively thick layer of asphalt and a relatively thick underlying layer of granular subballast. These thickened sections compensate for the absence of the ballast layer. The exact design and configuration of the ties, monolithic or two-block, slab track if used, and profile of the asphalt surface varies significantly as a function of preferential specifications. The application of cribbing rock, or some other means, is necessary to restrain the track from lateral and longitudinal movement. Its use in the U.S. has been restricted to a limited number of low volume highway crossings and small rail terminals and included yard tracks.

3.2 Asphalt Layer Thicknesses and Widths

The typical asphalt layer width is 12 ft (3.6 m) for open track, but it is placed wider under special trackwork, such as turnouts, crossovers, and bridge approaches to provide support under the longer ties.

The thickness of the asphalt layer depends on the quality of the roadbed's subgrade support and traffic loadings. A 6 in. (150 mm) thick layer is normally used for average conditions. For unusually poor roadbed support conditions, and for high-impact areas, a minimum of 8 in. (20 cm) is used. Ballast thickness normally ranges from 8 to12 in. (200 to 300 mm). A 6 in. (150 mm) thick asphalt layer that is 12 ft (3.6 m) wide requires 0.42 tons of asphalt per track foot (1.25 metric tons per track meter). Thicker asphalt layers are normally specified for WILDs and high impact special trackworks.

The asphalt layer should extend a reasonable length beyond the ends of the special trackwork so that subsequent track surfacing operations and any impact from track stiffness changes will not infringe on the area. The transition lengths may range from 10 to 25 ft (3.0 to 7.7 m) or longer.

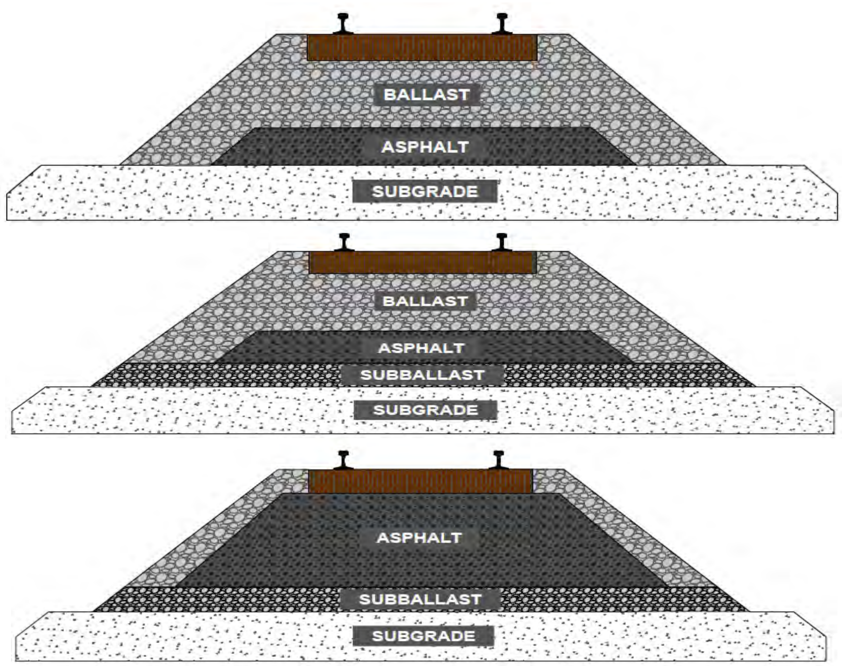

Figure 3.1 Asphalt Underlayment Trackbed (top), Combination Asphalt Trackbed (middle), and Ballastless Asphalt Combination Trackbed (bottom).

The roadbed should be reasonably well-compacted, well-drained, and capable of accommodating the hauling and spreading equipment without excessive rutting or deformation. A slight crown or side slope is desirable. Subsurface drainage or roadbed support improvements can be implemented prior to placing the asphalt if site conditions warrant, based on engineering evaluations.

3.3 Asphalt Mixture Designs

Recommended asphalt mix specifications, trackbed section designs, and application practices have evolved over the years. Slight variations from the initial mix designs and construction techniques are typical and have not affected trackbed performance. Asphalt trackbed design construction standard practices for railways typically follow recommendations set forth by the Asphalt Institute (8,9). Figure 3.3 shows an asphalt core taken from a trackbed.

The asphalt mix that has the ideal properties for the track structure environment is a low to medium modulus (plastic) mix, having design air voids of 1 to 3%. The mix will easily compact to less than 5% air voids in place. A local dense-graded highway base mix with a maximum aggregate size of 1.0 to 1.5 in. (25 to 37 mm) is typically specified.

Ideally, the asphalt binder content can be increased by about 0.5% above optimum for highway applications because rutting and bleeding of exposed highway pavement surfaces are not concerns in the insulated trackbed environment. This is similar to the bottom, or fatigue-resistant, asphalt layer of the perpetual pavement system as adopted for highway pavements in the U.S. The mix performance is significantly different in a trackbed application than in a highway application. Long-term monitoring and testing of in-service trackbeds indicate that this low voids, impermeable mix undergoes minimal oxidation from the effects of air and water. The mix is also isolated from extreme temperature fluctuation within the insulated trackbed environment (10).

The mix provides a layer having a reasonably consistent stiffness in hot weather and being slightly resilient in cold weather. Furthermore, compared to highway applications, in trackbed applications, this mix is much less likely to rut and bleed in hot weather and crack in cold weather, ensuring a long fatigue life.

Tests on subgrade/roadbed samples, obtained directly under the asphalt layer, indicate that the in-situ moisture contents are very close to optimum values for maximum density of the materials. For structural design analyses, it is reasonable to base bearing capacity values at optimum conditions for the soil/roadbed material under the asphalt layer (10).

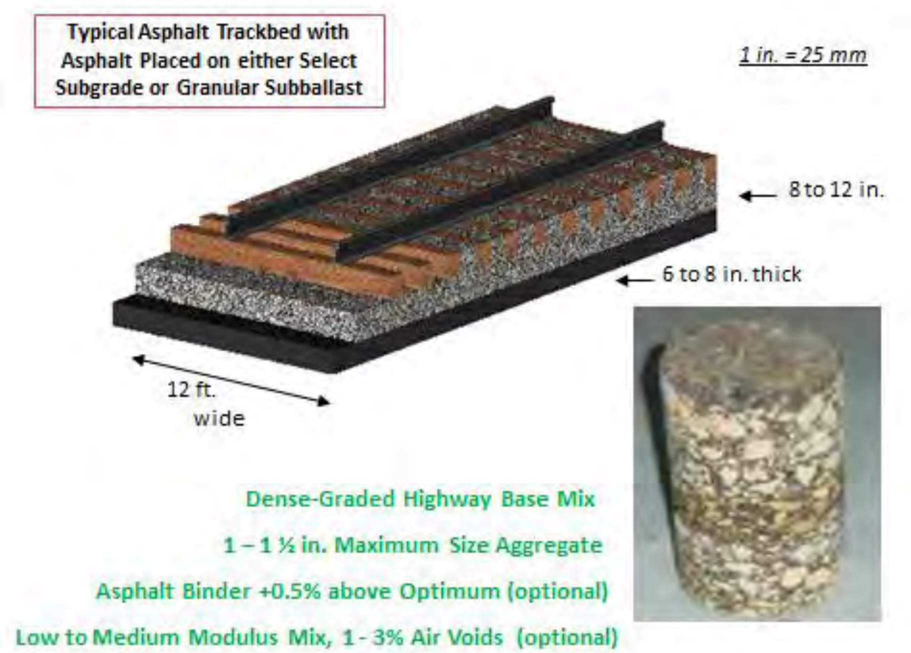

Figure 3.3 Configuration of Asphalt Trackbed and an Extracted Core.

SECTION 4 - SPECIFIC PROJECT DESIGNS AND PERFORMANCE EVALUATIONS

The applications described herein were taken from:

- Four large Class I freight railroads two in the eastern portion of the country – Norfolk Southern Railway and CSX Transportation, and two in the western portion of the country – BNSF Railway and Union Pacific Railroad,
- Two Class II railroads – Paducah & Louisville Railroad, a 270 mile (434 km) long Class II freight railroad in western Kentucky, and Portland & Western Railroad, a 520 mile (837 km) long Class II freight railroad and commuter rail line in western Oregon, and
- Three Urban Commuter/Light Rail Transit lines – Caltrain, a 55 mile (88 km) long line traversing the length of the San Francisco Bay Peninsular, Metrolink, a 534 mile (860 km) long line in the Los Angeles Basin, and the Denver Eagle P3 project, a 40 mile (64 km) long line.

The individual projects are described based on these types of applications -- Special Trackworks, Bridge Approaches, Tunnel Floors and Approaches, Wheel Impact Load Detectors, Middle-of - Street Trackage, Piedmont Improvement Program, and Open Track.

SECTION 5 - SPECIAL TRACKWORKS

Considerable research and development has been conducted to improve the performance of special trackworks – specifically rail-to-rail crossings, crossovers, and turnouts – to withstand heavy tonnages and axle loads and other high-dynamic loadings. Optimizing foundation stiffness is considered important for minimizing track settlement and minimizing track geometric variations; whereas, foundation damping is considered important for minimizing vertical dynamic loads, thus reducing the detrimental effects of high dynamic loadings (5).

These special trackworks are traditionally high impact areas due to the wheels traversing the flangeway gaps in the rails. Adequate drainage is often difficult to obtain, particularly in the switch point and frog areas. Also, the ballast is difficult to tamp and consolidate within the maze of rails and track components. Asphalt underlayment has been shown to increase trackbed strength while enhancing drainage thereby providing adequate and consistent support to obtain high ballast modulus to withstand and distribute the added vertical impact forces in the switch point and frog areas.

Normally, special trackworks have to be renewed "under traffic" during a short time period. Adequate planning is of utmost importance. It is even common to restrict the operations to nighttime and weekends, particularly on lines having commuter and passenger traffic. Equipment and personnel are selected to accomplish the project in a minimum amount of time. Normally, the track can be opened to traffic within 9 to 10 hours for a 4-diamond rail crossing. Single crossing and smaller size turnout replacements can be accomplished within 6 to 7 hours if properly planned. Minimal tamping or surfacing is required provided the ballast is pre-compacted on well-compacted asphalt subballast.

Literally hundreds of rail crossings, crossovers, and turnouts have been underlain with a mat of asphalt during the renewal and replacement of special trackwork. For example, CSX used asphalt underlayment for the replacement of numerous rail crossings in the Chicago area during the mid-1990s. CSX's B&O line east of Chicago had numerous rail crossings underlain with asphalt in northern Indiana and Ohio during the B&O double-track project. There are 12 rail crossings on the CSX and CSX/NS lines in Fostoria, OH underlain with asphalt (2,3).

Following are typical projects for turnouts and crossovers on four railroads.

5.1 Caltrain

This 55 mile (88 km) long regional rail link along the San Francisco Peninsula began using asphalt underlayment in 1999 for the rehabilitation/renewal of turnouts. During the intervening 18 years, 85 turnouts, predominately # 20s, at 24 sites have been rehabilitated/renewed using this technology. This number includes 13 of the 14 track crossovers on the main line. This mixed traffic line carries predominately frequent commuter trains and a limited number of UP local freight trains during the evening hours.

Construction details are contained in Caltrain's "Standard Drawings and Practices". An engineering evaluation is performed to determine unique conditions at each site that might influence the specific design. After removing the old turnout and roadbed materials, considerable attention is given to preparing the roadbed for the trackbed. This involves drainage considerations and thorough compaction of the existing or modified roadbed assuring a well-compacted 6 in. (150 mm) minimum thickness of compacted subgrade/roadbed.

The materials for the asphalt underlayment layer must conform to the provisions of Caltrain's "Standard Specifications". The underlayment mix is designated dense-graded Type A Mix with ¾ in. (19 mm) maximum size aggregate gradation. It is placed as a layer 8 in. (200 mm) thick, with either a machine or blade, and thoroughly compacted. The asphalt layer is placed 12 ft (3.7 m) wide and extends a minimum of 10 ft (3 m) along the track beyond the ends of the turnout. A layer of ballast, 9 in. (225 mm) thick, is placed above the asphalt.

The turnouts and crossovers have exhibited acceptable performance since installation. No subgrade support issues have been observed and the turnouts remain solid and serviceable. Based on demonstrated performance and engineering evaluations, the use of asphalt underlayment is considered the standard design for turnouts and crossovers when engineering evaluations indicate that the use of the technology is warranted. Figure 3 depicts a typical Caltrain crossover with asphalt underlayment.

5.2. Metrolink

This large commuter rail system in the Los Angeles area of Southern California presently consists of seven lines totaling 534 miles (860 km) in extent and continues to expand with additional lines. Metrolink began using asphalt underlayment in 2007 for turnouts, crossovers, and rail crossings. Since that time, numerous turnouts have been underlain with asphalt. It is considered standard practice for all newly constructed and rehabilitated turnouts. The design is similar to that specified by Caltrain, except that only 6 in. (150 mm) thickness of asphalt underlayment is used. The designs are contained in Metrolink's Standard Book of Specifications. A typical Metrolink crossover with asphalt underlayment is shown in Figure 5.1. The performances of these special trackworks have been satisfactory. Minimal settlement has been observed and they have remained solid and serviceable, requiring minimal maintenance.

5.3 NS Heartland

In 2012, NS replaced three No. 20 equilateral turnouts on the Heartland Corridor line near Kermit, WV. Each turnout was changed out during a 16-hour traffic curfew. The concrete turnouts were about 300 ft (90 m) long and pre-assembled in four sections. Two cranes were used to place the sections. The total time allocated for placing and compacting the 6 in. (150 mm) thick asphalt layer was 1¼ hours. The asphalt was placed with a typical paver in two 3 in. (75 mm) lifts. Views of one of the turnouts are shown in Figure 5.3.

Figure 5.1. Views of a Universal Crossovers on Caltrain at CP Palm (left) and on Metrolink's Antelope Valley Line (right). These Crossovers have been Maintenance Free for Many Years.

The performance of the three turnouts has been significantly improved during the past four years compared to the performance of the previous wood tie turnouts on existing trackbed/roadbed. No special surfacing has been required, just programmed surfacing every two years. Also no switch point failures, no guard rail adjustments with shims, no gage problems, and no pumping or fouled ballast have occurred and frog wear has been minimal.

Figure 5.3. A No. 20 Equilateral Turnout at Greyeagle, WV before positioning on the Asphalt Pad on left which had been placed earlier that morning and after four years of traffic on right.

5.4 Denver's RTD FasTracks – Eagle P3 Project

FasTracks is Denver's RTD (Regional Transportation District) voter-approved transit expansion program – the largest in the nation – transforming transportation through the Denver metro area.

This program, approved in 2004, augments the earlier completed light rail passenger lines serving the Denver Metro Area.

The Eagle P3 Project was approved in 2010. It includes the construction of three commuter lines as part of RTD's commuter rail line system. The project was delivered and is being operated under a concession agreement that RTD entered into with a "Concessionaire" known as Denver Transit Partners (DTP); the team is composed of several large companies. The Eagle P3 Project concession agreement requires DTP to design, finance, build, operate, and maintain the three lines -- A, G, and B, totaling 40 miles (64 km). The A & B lines opened in 2016; the G line is anticipated to open in 2017.

Of specific interest to this discussion was the decision by the DTP's design team to specify the use of asphalt underlayment designs for trackbed support for the turnouts and crossovers for the three new lines. The fact that the long-term maintenance agreement that DTP must adhere to for the next thirty years dictated that premium designs, materials, and construction techniques should be used to minimize subsequent maintenance expenditures.

There are numerous turnouts along the three routes. The specific asphalt underlayment trackbed design layer consists of 5 in. (125 mm) asphalt underlayment, placed a minimum 3 ft 3 in. (1.0 m) past the ends of the ties extending nearly12 ft (3.7 m) beyond the point of switches and the last long ties. The asphalt layer is topped with 12 in. (300 mm) ballast for the typical ballasted track.

Maintenance requirements and serviceability measures will be closely monitored during succeeding years for these three lines to determine the effects of the specific designs on the costs and warranty implications of the executed contractual obligations of the concessionaire. One of these studies will evaluate whether the additional cost of installing asphalt underlayments under turnouts will be recovered due to expected lowered maintenance costs and improved service metrics.

SECTION 6 - BRIDGE APPROACHES

Minimizing differential settlement and track stiffness variations at bridge approaches are paramount in maintaining requisite track geometric parameters to minimize impact stresses and excessive wear of the track components. Asphalt trackbeds are being used to achieve these qualities, documented examples follow:

6.1 Bridgeport, AL

This 1,475 ft (450 m) long heavy-tonnage bridge across the Tennessee River Slough was built in 1998 to replace an existing bridge. This required re-alignment of approximately 1,400 ft (425 m) of mainline track for both approaches.

Asphalt underlayment was selected to improve track substructure strength and reduce future maintenance. A 5 in. (125 mm) thick mat of asphalt was placed on a 6 in. (150 mm) thick granular subballast. Granite ballast 10 in. (250 mm) thick, concrete ties, and RE 136 CWR rail completed the track section on the two approaches. Figure 6.1 is a view of the asphalt underlayment and the finished bridge and north approach.

The CSX line also carries NS traffic. The total annual tonnage over this heavy tonnage and traffic line is about 70 MGT. During the 17 years since the bridge was opened to traffic, the approaches have required minimal track maintenance and the speed was increased from 10 to 30 mph (16 to 48 km/h). No mud or ballast fouling has been observed.

Figure 6.1. Core Drilling the Asphalt Layer in 1998, the day after the Asphalt was placed on the North Approach and a view of the North Approach in 2014, 16 years after Construction of the Bridge.

The native soil on the north approach was composed of highly plastic red clay with intermingled cherty aggregate. The native soil on the island was composed of silty sandy stream deposited material, much different from the clay soil on the Alabama approach. Both approaches, although constructed on wildly differing soil compositions, have performed admirably.

6.2 Ft. Estill, KY

Three short open deck bridges, just south of Richmond, KY, on CSX's Cincinnati to Atlanta mainline had habitually exhibited settlement, loss of ballast, and pumping near the bridge abutments. This situation required frequent maintenance to restore the track surface to minimize the effects on the ride quality of the locomotives and impact on the bridge and track approaches.

During a planned curfew on the line in the summer of 2001 the trackbed support materials on four approaches to three closely spaced open-deck bridges were replaced. For each approach, a 40-ft (12-m) long panel was removed, the old trackbed was dozed out and about 6 in. (150 mm) of asphalt was placed and compacted. Ballast was dumped on the asphalt and the panel was replaced. This process was accomplished in about four hours. Two of the approaches had not exhibited trackbed problems so they were not included in the four day blitz.

The use of asphalt underlayments for the three bridge approaches is considered a complete success. Recent inspections revealed no mud in the track and essentially no approach settlement after 15 years of service, in contrast to previous performances. Typically the crosslevel drops off slightly at the bridge abutments if the accompanying ballast shoulders are not adequately supported laterally. These approaches must be typically surfaced once per year while replacing ballast that has cascaded down the embankments, a low cost activity. Figure 6.2 shows views of the bridge during renewal and after 15 years of traffic.

Figure 6.2. Renewal process for Approach to a Short Open-Deck Bridge on CSX Line near Ft. Estill, KY (left) and condition after 15 years of Traffic (right).

6.3 Kentucky Dam and Muldraugh Hill

The Paducah & Louisville Railroad has used asphalt on approaches to several bridges; particularly bridges replaced requiring new trackbed approaches on different track alignments. Between 2004 and 2009 a new 3,100 ft (945 m) long bridge was built across the Tennessee River just below Kentucky Dam in far western Kentucky. The project was in conjunction with the widening and lengthening of the navigation locks at the dam site. Both the US 62 and P&L track were removed from the dam and relocated downstream on separate bridges.

The relocated alignment required a 4,000 ft (1,200 m) long, 75 ft (23 m) high compacted earth fill on the west approach which was constructed five years early to allow for settlement. The new west alignment on the fill and the east alignment, mainly in a cut for 2,000 ft (600 m) length, had 6 in. (150 mm) thick layers of asphalt topped with 12 in. (300 mm) of ballast. The wood tie trackbed abuts the ballast deck approach spans. The class 3 track has a 25 mph (40 km/h) speed limit across the bridge. Figure 6.3a shows views of the approach paving and the finished approach to the bridge.

The bridge was placed in service in 2009. Its yearly traffic tonnage is 7 MGT. In the intervening seven years the approaches have required essentially no track maintenance. The 2000 ft (600 m) long, 75 ft (23 m) high west end fill has shown minimal settlement, likely due in part to the waterproofing effect of the asphalt cap for shedding the rainwater.

Following the excellent performance of the approaches to the relocated Kentucky Dam Bridge, the P&L also chose to use similar trackbed designs for the approaches to two additional new bridges built off-line to replace two steel trestles. These 240 and 470 ft. (73 and 143 m) long concrete ballast deck bridges are near Louisville on the P&L's line on Muldraugh Hill. The bridges were built during 2012 and 2013. The asphalt trackbed approaches are 50 ft (15 m) long. Wood ties were used throughout both bridges and approach trackage. The approaches to the two bridges have required no track maintenance since opening to traffic. Figure 6.3b contains recent views of the west approach to the longer Muldraugh Hill Bridge.

Figure 6.3a. Paving the North End Approach prior to erecting the Bridge (left) and a recent view of the West End of the Kentucky Dam Bridge (right).

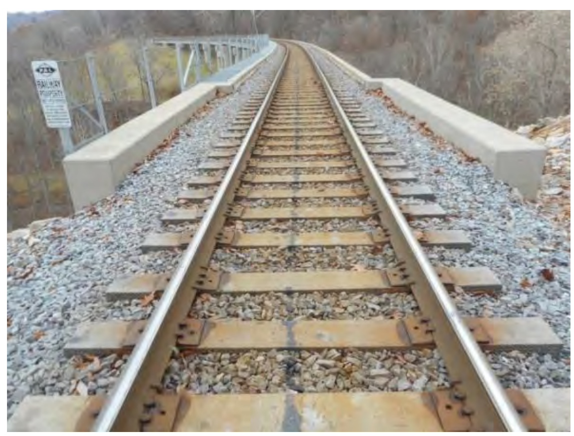

Figure 6.3b. Muldraugh Hill Bridge containing Asphalt Approaches that are 50 ft (15 m) long the full width of the wing walls for 25 ft (7.5 m) and an additional 25 ft (7.5 m). This particular Bridge contains Composite Ties; the other one contains Wood Ties.

6.4 Caltrain

Caltrain has used asphalt underlayment during the renewal of 16 approaches to bridges associated with the replacement of 8 bridges during the past eighteen years. It is considered a normal practice to place an 8-in. (200-mm) thick layer of asphalt while the track has been removed in preparation for installing new ballast and track to renew the track on the bridge approaches. The primary purpose is to minimize settlement and transition the track stiffness so that the ride quality at the bridge will not be compromised.

SECTION 7 - TUNNEL FLOORS AND APPROACHES

Maintaining a consistently high-quality trackbed support system in tunnels is vital for achieving optimum operating conditions and minimizing maintenance costs and track-induced slow orders. A properly designed and maintained trackbed system provides adequate support for the track and facilitates drainage. Maintenance costs and operational interferences are reduced, and higher levels of service and safety are attainable.

Intercepting and controlling drainage are highly important factors for achieving near maintenance-free tunnel trackbeds. Materials comprising many tunnel floors slake and weaken when they become wet. They are not capable of providing a uniformly stable support for the track. Pumping, ballast contamination, and associated track irregularities ensue, particularly on an all-granular trackbed, which is more subject to ballast/floor intermingling

Many tunnels have inherent geological drainage problems due to seeps or springs developing within the floor. These situations provide a constant source of water in the tunnel during wet weather. Water may flow out of the tunnel portals throughout most or all of the year. If the tunnel has a summit vertical curve, the drainage problem is usually less severe. Drainage can flow out both tunnel portals.

Drainage around portal areas should be adequately planned for and maintained. Surface drainage must be collected and prevented from entering the portal area. Approach ditches, pipes, and inlets must be kept clear of debris and maintained free flowing away from the portal. Drainage that is backed up within the tunnel trackbed provides a primary source for track instability problems, resulting in subsequent deterioration of the track surface and alignment.

Premium trackbed systems proposed for tunnels to minimize the detrimental effects of poor quality (soft) floor support and inadequate drainage typically involve placement of a solid layer or slab of a near impervious material within the track structure. Direct fixing of the rails to a slab of concrete or other rigid material is used. Consistent support and proper dampening of impact forces must be achieved. These systems are typically more expensive than the open ballast trackbed system.

During the past several years, asphalt has been used successfully to rehabilitate numerous tunnel trackbeds, which were exhibiting high maintenance costs due to poor quality trackbed support and inadequate drainage. The asphalt provides an impermeable, semi-rigid underlayer to fill in the low spots and provide an asphalt thickness that may vary from 0 to 4-6 in. (0 to 100-150 mm) above the prevailing tunnel floor. Minor track surface adjustments can be made in the ballast.

Typical rehabilitation procedures, while maintaining traffic, involve first removing the equivalent of 3 to 4 track panels from within the tunnel and for a specified distance outside the portal. The contaminated ballast/floor material is excavated to the desired level, preferably to a reasonably dry, solid bed. The asphalt is hauled by dump truck from a hot mix plant and is either spread with a highway paver or, as is more common in tunnels, merely back-dumped and spread with a loader bucket or with a dozer blade. Close grade control is not required because the layer of ballast will serve as a leveling course. Rolling and compaction of the mat follows.

The track can be immediately dragged back on the asphalt mat and joined to the existing track prior to unloading ballast. An alternate procedure is to dump a layer of ballast on the asphalt mat prior to dragging the track to final position. Final ballast application and surfacing follow to achieve the specified top-of-rail elevation. The process is repeated during the following days to effectively change out 120 to 160 ft (36 to 48 m) of track per day.

The asphalt mat should extend the full width for the typical 12-ft (3.6 m) wide tunnels. Provisions can be made for longitudinal perforated pipes along the tunnel walls to facilitate collection and drainage of water. Asphalt thickness is often limited by vertical clearance requirements. It often ranges from 1 in. (25 mm) to possibly 10 in. (250 mm) at low spots. The average thickness is typically 4 in. (100 mm). Since the major purpose of the asphalt mat is to level the floor, the thickness will necessarily vary considerably.

7.1 CSX Transportation

During the mid-1990s, CSX Transportation rehabilitated all or portions of nine tunnels on mainline in the eastern Kentucky/Tennessee area. Each one had historically been a "wet" tunnel exhibiting similar characteristics – soft support and inadequate drainage in low areas which "ponded" water contributing to rapid loss of acceptable track geometry. This adversely affected track geometry resulting in slow orders, excessive maintenance costs, and operational interferences. Previous efforts, such as undercutting the track and adding various fabrics had not been considered effective.

The performance of these tunnels during the intervening 15 or more years has been significantly improved. CSX has utilized this procedure for additional tunnels. The prevailing problem is being able to obtain an adequate time frame to accomplish the work. Normally a 10- to 12-hour curfew is necessary for changing out an equivalent of 3 to 4 panels. Recent views for two of the tunnels after 19 years since paving with asphalt, north of Knoxville, TN, are shown in Figure 7.1.

Figure 7.1. Vasper Tunnel (left), 1500 ft (457 m) has been Dry all the way through only requiring maintenance surfacing for crosslevel twice during Five-Year Spans. Similar success for the Cove Lake Tunnel (right), 1750 ft (533 m) except for a couple of waterfalls requiring maintenance to periodically clean the ditches. Both are on the KD Subdivision.

7.2 Norfolk Southern

More recent tunnel projects involve the final three tunnel clearance improvement projects on the Norfolk Southern line just west of Williamson, WV during August 2010. This was part of the Heartland Corridor capacity improvement project. Clearances were increased in 28 tunnels between Norfolk, VA and Columbus, OH to accommodate double-stack intermodal trains. This was a two to three-year long project. Most of the tunnels were amenable to removing sufficient roof material to achieve the required clearances. This was accomplished using four approximate 12-hour track curfews each week.

However, three of the single-track tunnels, totaling 8,098 ft. (2,468 m) long, in close proximity to Kermit, WV had the tunnel floors lowered to achieve most of the vertical clearance. These three tunnels had a long history of soft support and attendant drainage problems requiring frequent maintenance interfering with normal train operations on this mainline. The decision was made to do this work during a 72-hour total shutdown of the line as the last major tunnel clearance activity.

Three contractors were used to perform the work simultaneously on the three tunnels. The initial 24-hour period was used to remove the track and excavate/remove sufficient floor material to provide space for the approximate 6 to 8 in. (150 to 200 mm) thickness of asphalt. This involved hydraulic hammers to remove portions of the floor. The intermediate 24-hour period was used to place, level, and compact the asphalt. About 7,000 tons (6,350 metric tons) of asphalt was used for the 6,543 ft (1,994 m) of tunnel length for the three tunnels plus the tunnel approaches. The final 24-hour period was used to replace the ballast and track panels. The work was finished within 68 hours and opened on September 1 as planned. Figure 7.2 shows the asphalt mat.

The three tunnels are performing extremely well after six years of train traffic. At a few sites, mud will seep along the seam of the asphalt floor and the tunnel wall. It is still difficult to maintain drainage along the ditches. When this is impaired, mud will migrate to the track. The performance of all three tunnels is considered to represent major improvements compared to pre-Heartland performance prior to placing the asphalt and increasing the clearances. Pumps were re-installed in one of the tunnels; which requires periodic ditch maintenance when the pumps fail to operate adequately.

Figure 7.2. Big Sandy Tunnel No. 3 near Greyeagle, WV with the paved trackbed during the Tunnel Clearance Project in 2010 and the finished approach to the Tunnel in 2016. Maintenance costs have been minimal since 2010.

7.3 <u>Caltrain</u>

Within the past eighteen years Caltrain has used asphalt underlayment to correct historically poor support and improve drainage in its four tunnels just south of San Francisco. This predominately rail commuter line accommodates daily upwards of 75 commuter trains and additional freight trains. The approaches to all four tunnels have asphalt underlayments and the inverts to two of the tunnels have asphalt underlayments throughout the tunnels. The performance of the trackbeds has been substantially improved and the remaining two tunnel inverts are scheduled to be paved with asphalt. Figure 7.3 contains views of two of the tunnels containing asphalt underlayments. .

Figure 7.3. Two of the Caltrain Tunnels paved with Asphalt just south of San Francisco. Tunnel 4 (left) and Tunnel 2 (right.).

SECTION 8 - WHEEL IMPACT LOAD DETECTORS

A major consideration for the track support structure for WILD installations is that it be reasonably stiff and maintain a consistent stiffness along the length of the installation throughout the year. A main factor influencing the maintenance of consistent track stiffness is maintaining consistent moisture content of the subgrade/subballast layers throughout the year.

The ballast layer must have consistent and uniform support so that it can develop maximum density/compaction to behave linearly elastic achieving maximum shear strength for distributing pressures uniformly, but still maintain a reasonable degree of elasticity. These factors will minimize rail deflection and track galloping thus providing a smoother ride with less vibration and deflection.

In recent years the use of a layer of hot-mix asphalt, similar to highway paving mixtures, has gained widespread acceptance as a subballast to provide the desirable attributes of the support layer for the WILD's support track and ballast. The reasonably stiff asphalt layer also serves to basically waterproof the underlying subgrade layer. Ideally, a subgrade soil should maintain a uniform moisture content at or slightly above optimum throughout the year ensuring consistent support along the instrumented test area.

As the subgrade moisture content increases above optimum, the strength and rate of deformation under repeated loading increases with attendant loss of strength and load carrying capacity. The resulting increased deformations and abrasion tend to degrade the ballast by producing mineral fines which vary the stiffness and support characteristics of the ballast. Pumping and loss of track surface elevation levels result in uneven ride quality and increased and variable impacts resulting from the variable support conditions.

The objective is to specify and construct a track support structure that provides consistent support for the track on which the trains will be traversing. Therefore, variations in test data will be indicative of the effect due to wheel-rail interface surface abnormalities affecting the impact measurements.

Class I railroad companies have been actively involved with the installation of a layer of asphalt under WILDs for several years. Actually, Conrail, prior to its dissolution, was using asphalt under WILDs some 25 or so years ago. NS and CSX inherited some of these.

8.1 Norfolk Southern

During the past several years, NS has installed asphalt underlayments under twelve WILDs at nine locations. A typical installation is shown in Figure 8.1. These are specified for new installations and for rehabilitating previously installed WILDs. Asphalt underlayment is a standard practice for all WILDs. It is specified as a matter of initial track design for new installations and it is added when performing major renewal or rehabilitation of existing WILDs.

The specified dimensions of the asphalt layer vary from site-to-site depending on the prevailing conditions. Typically the layer is 12 in. (250 mm) thick and 12 ft (3.6 m) wide and extends a distance of 150 ft (46 m) or longer.

8.2 CSX Transportation

During the past few years, CSX has installed asphalt underlayments at majority of the WILD sites. These include WILDs at Supersites that contain additional trackside measuring and detection equipment, and WILD-only sites. CSX's typical WILD track section containing asphalt underlayment was issued in 2006. It specifies a layer of asphalt underlayment 6 in. (150 mm) thick and 12 ft (3.6 m) wide, crowned and sloped 2% transversely, topped with 12 in. (300 mm) of ballast under the concrete ties. Figure 8.2 depicts a CSX WILD installation. The WILDs with the asphalt underlayments have served satisfactorily eliminating mud, pumping, and wavy track within the measurement area.

Figure 8.1. Asphalt Paving in preparation for installing the Salient Process WILD on NS Line at Flatrock,KY in 2009.

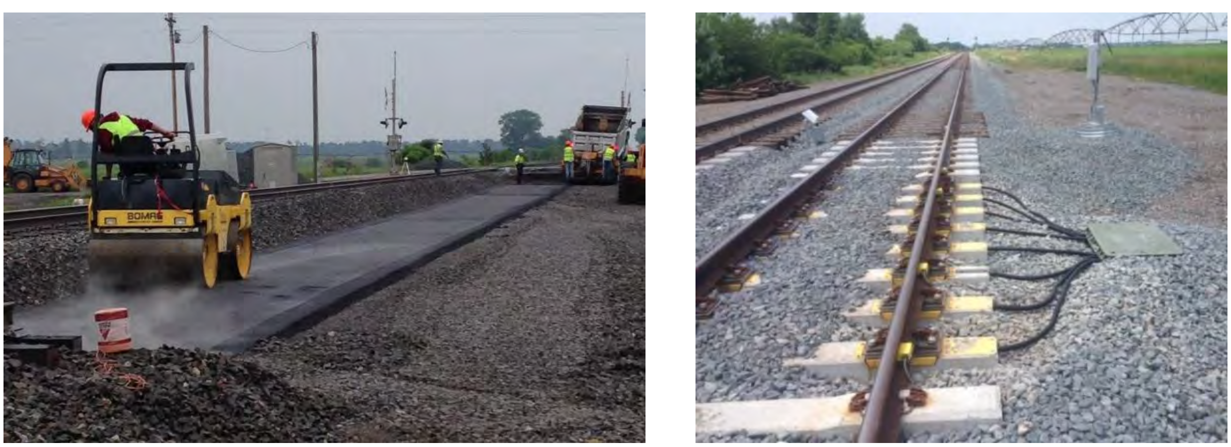

Figure 8.2. Asphalt Paving in preparation for Installing the Schenck Process WILD on CSX Mainline at Webster, IN.

SECTION 9 - MIDDLE-OF-STREET TRACKAGE

A unique type of a railway/street jointly shared facility is the situation where the railroad track occupies the middle of the street for a considerable length. This is basically a continuously closed crossing so that street vehicles have continuous access beside and across the track. Typically on-street parking is permitted in urban areas if the street is sufficiently wide to accommodate parking.

Drainage from within the crossing is difficult to achieve and maintain at these locations which can adversely affect the load carrying capability of the track. The track can settle non-uniformly resulting in a wavy track negatively impacting the track geometry and street ride quality.

Access to the track to perform maintenance is time consuming and expensive and impacts normal train and street traffic. The crossing surface and surrounding surface approaches must be removed and replaced. It is very desirable for mid-street closed tracks to have proper structural support below the track so that the track and street components do not settle with attendant roughness requiring frequent removal and renewal.

9.1 Portland & Western in Oregon

Of particular significance to the P&W is the predominance of long-distance middle-of-street running trackage in several cities. This was common to the predecessor Oregon Electric Line that operated frequent interurban passenger trains in the southern Portland suburbs. Crossings ranging from 2,000 to 3,500 ft (600 to 1,100 m) long are common. These have typically required frequent and costly track maintenance activities to maintain smooth surfaces for both train and highway vehicle passage. This has included frequent surface replacements, roadbed injection, pavement milling, etc. Providing adequate drainage throughout these long distances to minimize settlement and deterioration and provide adequate trackbed support for the jointly used crossings has been difficult to achieve, for selected crossings, using conventional all-granular support structures.

Renewing the crossing surfaces and support structures for the poorly performing middle-of-street crossings began in 2009 in Independence with a 2000-ft (600-m) long Main Street crossing installed over four years using asphalt/rubber seal surfaces with concrete at selected cross streets.. This was followed with a 3,500-ft (1,100-m) long Holly Street concrete crossing in Junction City, also installed over four years. The P&W trackage has 2.9 miles (4.7km) of middle-of-street-running crossings, scattered over six cities. Recent views of the two crossings are shown in Figure 9.1.

Performance of the two crossings to date has been very satisfactory. During 2017 P&W plans similar projects in two cites – 2,800 ft (850 m) long Front Street in Salem and 1,400 ft (425 m) long 4th Street in Harrisburg. These will be multi-year projects following the specifications and procedures used for the initial two long-distance street crossings.

Figure 9.1. Two of P&W's Middle-of-Street paving projects utilizing Asphalt Underlayments. The 3,500 ft (1,100 m) long Holly Street Trackage in Junction City (left) and the 2000 ft (600 m) long Main Street Crossing in Independence (right).

9.2 NS in West Brownsville, PA

NS recently completed the renewal of a 3300 ft (1,000 m) long crossing on Main Street in West Brownsville, PA. NS renewed this crossing in four sections, each about 800 ft (244 m) long over a four-year period during 2011 to 2014 to rectify a previous chronic maintenance expense due to having to renew portions of the crossing at frequent intervals. The final one-fourth of the crossing was renewed during a maintenance blitz on this line in 2014. The crossing, located on the heavy tonnage coal-hauling Monongahela line, has asphalt underlayment support and a concrete surface utilized along the entire distance during the 2011–2014 renewals. Previously the trackbed and crossing surface were being renewed every four years using conventional all-granular subballast support.

A longitudinal trench 13 ft (4 m) wide was excavated and a 4 in. (100 mm) thickness of granular subballast was added topped with an 8 in. (200 mm) thick layer of asphalt that replaced a portion of the all-granular subballast. A 12 in. (300 mm) thickness of ballast was placed and compacted. Wood tie track panels were positioned on the compacted ballast. The addition of typical concrete crossing panels completed the installation within the trackbed. Figure 9.2 shows the crossing during the renewing and the south end after five years of service.

The crossing is performing very well since the adoption of the asphalt support. The oldest section has been in service for five years. The crossing is showing no indication of distress or deterioration and all indications that it will serve adequately for many more years.

Figure 9.2. Main Street Paved Trackage in West Brownsville, PA. The earliest placed section after five years of traffic (left) and paving one of the sections within the excavated trench (right).

SECTION 10 - PIEDMONT IMPROVEMENT PROGRAM

The Piedmont Improvement Program (PIP) is a largely federally-funded railway/highway project in the Piedmont area of North Carolina. The primary project involves a portion of the Southeast High Speed Rail Corridor between Greensboro and Charlotte, also part of the Norfolk Southern Railway's Crescent mainline. The actual railway is owned by the North Carolina Railroad, a state owned entity, and the project is primarily administered by the North Carolina Department of Transportation and the Norfolk Southern Railway Company. The primary purpose of the project is to increase passenger and freight rail capacity, increase train speeds, and improve railway and highway safety.

A primary aspect of the project is to add 26 miles (42 km) of second track between Greensboro and Charlotte to complete the double-tracking of that 92 mile (148 km) portion of the Southeast High Speed Rail Corridor. Passenger train speeds on the Crescent line will be increased from 50 mph (80 km/h) to 60 mph (96 km/h) on diverging moves through the No. 24 crossovers. Two additional round trip passenger trains will be added increasing the Charlotte to Greensboro line, continuing to Raleigh, to ten daily trains. Also, the line can accommodate increased numbers of higher-speed freight trains on the higher capacity line.

For many years this section had portions of double track with three interspersed single track portions. These required the use of a series of lateral turnouts for the trains to change from single to double track on the 52 MGT heavy traffic line. Five new No. 24 high-speed concrete tie double crossover special trackworks were installed to provide the capability to direct traffic from one track to the other as required to achieve the increased level of operating efficiency to accommodate increased numbers and speeds of trains. The decision was made to utilize asphalt trackbeds for the 20 turnouts at the five double crossovers.

The trackbed design for the five double-track crossovers included a 6 in. (150 mm) thickness of crushed stone subballast on the prepared subgrade topped with a 6 in. (150 mm) thickness of asphalt meeting NCDOT section 610 specifications. The asphalt extended for a continuous 1,135 ft (346 m) length to include the four turnouts. The same design was used for transitions extending additional 125 ft (38 m) lengths beyond the point of switch on both ends for a total length of 1,385 ft (422 m) for each double crossover. A 12 in. (300 mm) thickness of ballast was placed above the asphalt. The existing track was shifted as necessary to maintain traffic so that the asphalt could be placed under the existing track and the new track achieving a 29 ft (8.8 m) wide continuous asphalt pad under the two tracks while maintaining 14 ft (4.3 m) wide track centers. The logistical plan was designed to minimize delays to existing traffic while the asphalt paving and track shifting were underway.

A new WILD near China Grove was placed on No. 1 track. The track design included a 12 in. (300 mm) thickness of crushed stone subballast on the prepared subgrade topped with a 12 in. (300 mm) thickness of asphalt and 12 in. (300 mm) thickness of ballast. The asphalt pad was placed 12 ft (3.6 m) wide and 400 ft (122 m) long. The existing WILD on No. 2 track was removed and re-installed using the same track structure design as the new WILD.

The crossovers and WILDs are performing perfectly. The structural aspects of the trackbed appear to provide ideal combination of stiffness and flexibility for smooth passage of wheels over the rail flanges in the crossovers and provide consistent support along the measurement length of the WILD. Figure 10 shows construction views of the asphalt underlayment for one of the crossovers, a completed crossover at CP Lake, and the two WILDs near China Grove.

SECTION 11 - OPEN TRACK

Asphalt trackbeds are selectively utilized for numerus open-track applications. Many of these have involved reasonably routine double-tracking and capacity improvement projects consisting of new roadbed construction for variable lengths. Therefore the asphalt can be rapidly and efficiently placed with conventional asphalt paving equipment normally used for highway paving.

Additional projects involve rehabilitating short sections of in-service tracks to enhance load carrying and drainage properties of the trackbed support. Depending on the lengths of the sections rehabilitated, track closures can range from 6 to 18 hours with adequate pre-project planning and successful project execution.

Following are description for two significant triple-track capacity improvements projects on the BNSF/UP railroads in the central portion of the U.S. Both of these required maintaining traffic during the construction processes.

Figure 10. Compacting the Asphalt Layer and the finished Asphalt Underlayment Layer in preparation for placing a No. 24 Crossover (top), a Finished Crossover at CP Lake and the Two WILDs near China Grove (bottom).

11.1 <u>BNSF/UP Central Corridor Project in Wichita, KS</u>

A multi-benefit application for asphalt trackbeds was incorporated into the design of the Wichita Central Corridor Railroad Grade Separation Project. This 4 1/2- year long project, recipient of national recognition and several awards, was completed in 2009. Two miles (3.2 km) of track was raised above the roadway providing five new bridges to carry trains over the five arterial streets below. Between the bridges, the tracks were elevated on a 25 ft (7.6 m) high embankment supported by precast concrete T walls. The primary benefits were to mitigate issues related to highway traffic congestion, delays, and vertical clearances for trucks while minimizing train-induced noise. The safety aspects for both highway and train traffic were enhanced by improvements provided by elevating the train tracks above the street and providing enhanced clearances for the street traffic. The elevated track also served to minimize noise from the 40-plus daily trains. Context sensitive solutions were incorporated throughout the project.

Various geotechnical innovations were used to insure that the embankment fill was sufficiently competent to withstand the loadings with minimum settlement of the fill material and underlying native materials. This included installing 3,000 stone columns ranging in length from 2 to 27 ft (0.5 to 8.2 m). The existing foundation materials consisted of highly variable sand and clay fill overlying various thicknesses of compressible silt and clay soils as deep as 27 ft (8.2 m).

The reinforcement measure used for the embankment permitted the use of economical, locally available sand for the embankment material. This was topped with a graded granular layer consisting of the salvaged ballast that met the required gradation for granular subballast. However it was deemed necessary to minimize water intrusion into the marginal quality embankment material to insure that its structural properties were not compromised inducing settlement and increased pressures on the retaining walls.

These issues were addressed by placing a layer of impermeable asphalt over the graded granular layer. The asphalt was placed 6 in. (150 mm) thick for the total width of 66 ft (20 m) between the vertical walls. The asphalt extended from the transition from the at-grade existing track at the north end to the tie-in with the existing elevated track 2.5 miles (4 km) south near the former train station. The asphalt was topped with 12 in. (250 mm) of mainline-grade ballast. The numerous surface mounted inlet grates were designed to intercept and convey rain water to drainage pipes that exit as weep holes near the bottom of the retaining walls. The paved trackbed is sufficiently wide to accommodate three tracks, although only two tracks were installed initially to handle existing railway traffic. Figure 11.1 shows construction views and the finished trackbed.

The performance of the paved trackbed over the past seven years has been excellent. This is viewed as a specific benefit of the asphalt layer by increasing the load carrying capabilities of the trackbed and decreasing water intrusion into the underlying trackbed support. No specific track maintenance has been required other than normal programed surfacing. The track geometry has consistently maintained conformance to the FRA requirements for Class 4 track with no track-related slow orders.

The traffic flow on the arterial streets has improved substantially. The improved bridge horizontal and vertical clearances provide for improved safety for street vehicles and marked improvement in aesthetics of the surrounding areas. The train induced noise has been significantly reduced which can be attributed to the sound-attenuating effects of the dissimilar graduated granular fill and the somewhat viscous layer of asphalt serving as a vibration damping and attenuation medium. Additional noise damping is attributed to the protruding vertical walls atop the T-walls and the elimination of train horns throughout the grade-separated corridor.

11.2 BNSF/UP Capacity Improvement Project near Eton, MO

An interesting application of an asphalt trackbed, based on engineering evaluation, was for a short section of an 8-mile (12.9 km) third track addition and curve reduction capacity improvement project on the BNSF Transcon Mainline east of Kansas City, MO. The trackage along the Missouri River between Congo and Eton Junction, near Eton, contained this section of trackage that had historically required frequent track maintenance. Track surfacing was extensively utilized, at times nearly continuously, to maintain required geometric parameters. This was due to the native shale in a cut area exhibiting poor quality engineering properties. In addition, there were several 6 degree horizontal curves that required 35 mph (56 km/h) speed restrictions on this very heavy traffic 70 mph (113 km/h) BNSF and UP combined trackage line.

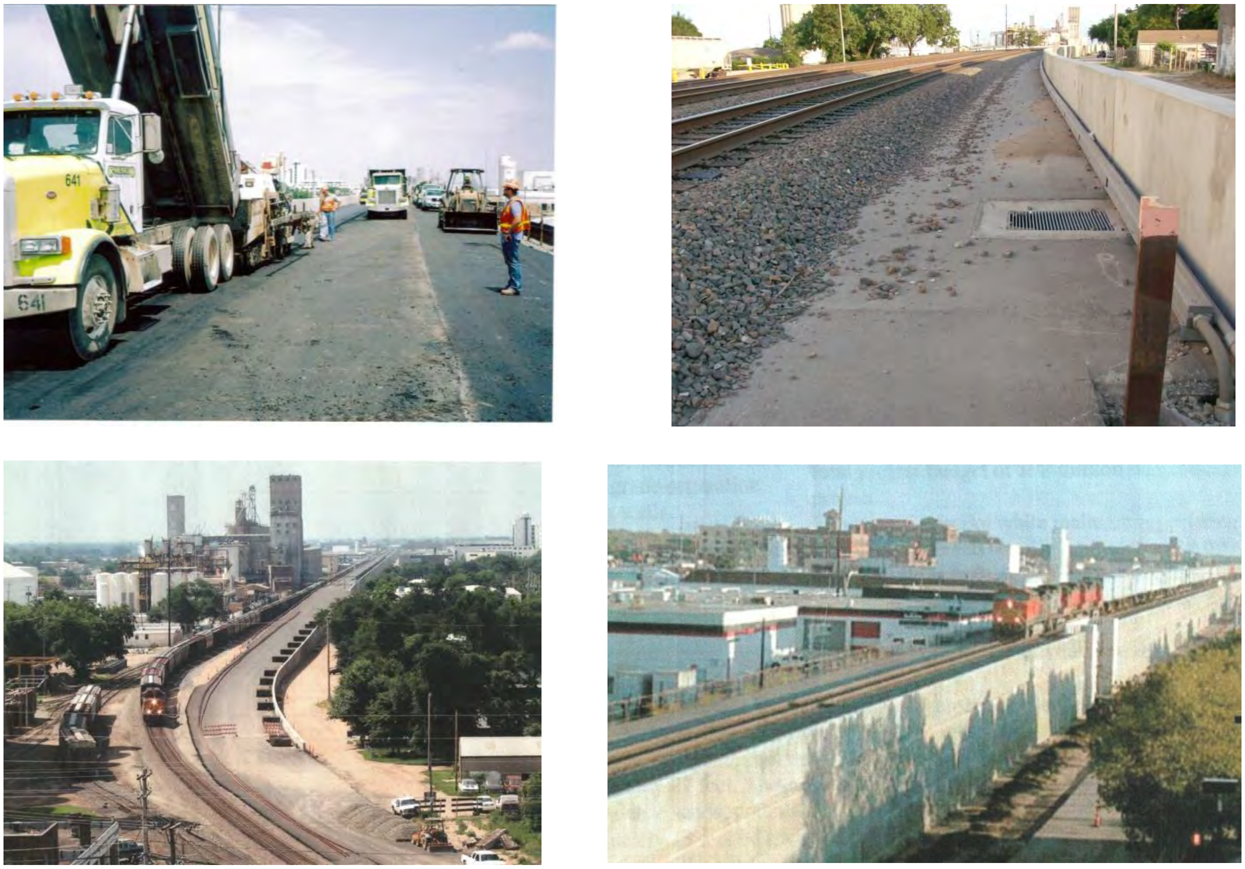

Figure. 11.1. Paving the Elevated Trackbed and the Finished Trackbed are shown in the top views. The bottom views show the exposed asphalt on the north end of the Elevated Track prior to placing the track and the precast concrete T-Walls and Mechanically Stabilized Earth Walls.

Studies indicated that a third track was needed to increase capacity for efficiently accommodating the projected increased BNSF and UP traffic in this area. Also a solution was sought to mitigate the adverse performance of the shale, assumed to be an even more significant problem with newly exposed materials. In addition, the horizontal curves resulted in reducing train capacity and performance while accruing added track maintenance costs which included frequent curve rail replacement. There were also some environmental constraints relative to impacting restricted areas within the flood plain that could be disturbed and encroachment restraints on previous environmentally sensitive areas. This required considerable excavation in the adjoining hillsides to move the extra track closer to the hillside and farther from the river.

Therefore the decision was made to decrease the curvature of several horizontal curves to straighten the horizontal alignment to improve operating performance during the construction of the third track. In addition an asphalt trackbed was selected to minimize the adverse effects of the poor performing native shale material through the cut for 900 ft (275 m). As soon as the subgrade was prepared, the grade was immediately paved with a 6 in. (150 mm) thickness of asphalt.

This was considered necessary to minimize the high shrinkage values and associated loss of strength of the shale material since it slaked upon exposure to weather, resulting in excessive cracking and water absorption. This process was performed while maintaining traffic on the existing line. The use of asphalt on other non-shale segments of prepared subgrade was evaluated, but the abundance of on-site limestone available for crushing resulted in a 12 in. (300 mm) thickness of aggregate subballast sections being significantly less costly than a 6 in. (150 mm) thickness of asphalt for the remainder of the project.

Asphalt was placed under all three tracks for a distance of 900 ft (275 m) from MP 437.45 to 437.62 during the period in April, 2010. Figure 11.2 contains views of the construction process and the finished track in service.

During the six years since the re-aligned track and third main have been in service the section of track over which the asphalt underlayment was used has shown no instability due to subgrade problems. The only maintenance required is regular surfacing. No geometry defects have been noted. The use of the asphalt underlayment appears to have mitigated possible trackbed instability problems due to constructing the re-aligned track over the native shale material.

SECTION 12 - CONCLUDING COMMENTS

The United States railway industry continues to emphasize the importance of developing innovative trackbed design technologies for both heavy tonnage freight lines and commuter/transit passenger lines. The purposes are to achieve high levels of track geometric standards for safe and efficient train operations while minimizing long-term track maintenance costs and extending track component service lives.

During the past several decades designs incorporating a layer of asphalt (or bituminous) paving material as a portion of the railway track support structure have steadily increased until it is often considered as a common or standard practice. Asphalt trackbeds have been primarily limited to heavy tonnage freight lines and commuter/rail transit lines in the United States, most often for maintenance/rehabilitation of special trackworks – such as turnouts, rail crossings, highway crossings, WILDs, tunnel floors, bridge approaches, etc., or capacity improvements of existing lines. Based largely on its proven performance it is considered a standard engineering design practice for specified types of applications on a variety of freight railroads and commuter/transit

Figure 11.2. Construction views (top) in 2009 showing the Asphalt Paving through the shale cut for the curve reduction project. Finished views (bottom) of the re-aligned track through the shale cut; 2011 view on left and recent view on right.

rail lines in the United States. This technology also has demonstrated applications for the construction of numerous new high-speed passenger lines in Europe and Asia.

The asphalt layer is normally used in combination with traditional granular layers to achieve various component configurations. This practice augments or replaces a portion of the traditional granular support layers and is considered to be a premium trackbed design. The primary documented benefits are to provide additional support to improve load distributing capabilities of the trackbed layered components, decrease load-induced subgrade pressures, increase confinement for the ballast, improve and control drainage, maintain consistently low moisture contents in the subgrade, insure maintenance of specified track geometric properties for heavy tonnage freight lines and high-speed passenger lines, and decrease subsequent expenditures for trackbed maintenance and component replacement costs.

The projects described herein were all designed and installed using highly technical standards of engineering practice relating to the developed technology for using a layer of asphalt, termed "underlayment", within the track structure to achieve specified high standards for trackbed performance. This factor is largely responsible for the perfect to near-perfect performances for all of the forty projects included and evaluated in this study. The coverage reported herein is not all-inclusive, but represents typical, pertinent, and unique applications of which the author is familiar.

SECTION 13 - ACKNOWLEDGEMENTS

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A Laboratory Test Method for Measuring Realistic Trackbed Pressures at the Tie/Ballast Interface

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TECHNICAL SUMMARY

Title

A Laboratory Test Method for Measuring Realistic Trackbed Pressures at the Tie/Ballast Interface

Introduction

This report documents the development of a practical and reliable laboratory method for calibrating pressure cells and quantifying typical railway trackbed pressures at the tie/ballast interface under simulated trackbed conditions. Vertical pressures were applied by a hydraulic compression test machine to prototype sections of the trackbed consisting of a section of rail placed on a tie plate and a section of a wood tie, supported by specially designed frames/boxes containing the ballast and subballast.

Approach and Methodology

The granular material pressure cells were recessed within the bottom of the ties, thus suitably positioned for subsequent in-track measurements. Loading magnitudes, applied by the testing machine, and resulting vertical interfacial pressures were similar to those exerted by locomotives and loaded freight cars traveling at typical freight train speeds.

Particular attention was given to evaluating the effects of various procedures for positioning the cells within the interface and attaching the cells to the ties. Additional attention was given to optimizing the design of the ballast and subballast containment frames and testing procedures so that the stress dissipation would simulate in-track loadings typically produced by freight trains.

Findings

A granular material pressure cell, manufactured specifically to measure pressures of large-size graded aggregates, accurately measured vertical pressures under laboratory simulated in-track loading using known and controlled loadings. Near perfect correlation (R^2 of 0.9997) was obtained between repeated machine applied pressures and measured cell pressures.

Test data indicated minimal variations between:

- Measurements using different cells for identical test conditions,
- Measurements for cells positioned below a solid tie (not practical for in-track applications) and cells recessed in the bottom side of the tie, and

- Measurements with cells "floating" loose within the recesses, cells attached with screws or cells attached with corner braces exhibited minimal variations.

Conclusions

The effects of various alternate procedures were evaluated. The selected procedure provides excellent correlation between the controlled applied machine pressures and the simultaneously measured cell pressures. The selected granular material pressure cell is considered applicable for in-track tie/ballast pressure measurements using the techniques developed in this study. However, the specific procedures utilized to install and position the cells in the trackbed can adversely affect the associated pressure measurements. Following proper techniques for installing and positioning the cells are critical for obtaining realistic pressure values.

Recommendations

For future in-track tie/ballast interface pressure measurements, the instrumented ties should be recessed by routing to provide space to embed and protect the active cell, transducer housing, and instrument cable.

The cells should be attached to the ties with either screws or corner braces and protected from damage.

Publications

Rose, J.G. Clarke, D.B. Liu, L. and T.J. Watts. "Development of a Laboratory Test Method for Measuring Trackbed Pressure at the Tie/Ballast Interface". Paper 18-00592 TRB 97th Annual Meeting Online. Transportation Research Board, January, 2018, 12 pages.

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SECTION 1 INTRODUCTION

This report documents the development of a practical and reliable laboratory method for calibrating pressure cells and quantifying typical railway trackbed pressures at the tie/ballast interface under simulated trackbed conditions. Vertical pressures were applied by a hydraulic compression test machine to prototype sections of the trackbed consisting of a section of rail placed on a tie plate and a section of a wood tie, supported by specially designed frames/boxes containing the ballast and subballast.

The granular material pressure cells were recessed within the bottom of the ties, thus suitably positioned for subsequent in-track measurements. Loading magnitudes, applied by the testing machine, and resulting vertical interfacial pressures were similar to those exerted by locomotives and loaded freight cars traveling at typical freight train speeds.

Particular attention was given to evaluating the effects of various procedures for positioning the cells within the interface and attaching the cells to the ties. Additional attention was given to optimizing the design of the ballast and subballast containment frames and testing procedures so that the stress dissipation would simulate in-track loadings typically produced by freight trains.

The effects of various alternate procedures were evaluated. The selected procedure provides excellent correlation between the controlled applied machine pressures and the simultaneously measured cell pressures. The selected granular material pressure cell is considered applicable for in-track tie/ballast pressure measurements using the techniques developed in this study. However, the specific procedures utilized to install and position the cells in the trackbed can adversely affect the associated pressure measurements. Following proper techniques for installing and positioning the cells, as described herein, are critical for obtaining realistic pressure values.

SECTION 2 PREVIOUS STUDIES

It has been desirable for years to develop a reasonably simple, accurate, and reliable method to directly measure pressure magnitudes and distributions at the tie/ballast interface in railroad trackbeds. Quantifying the magnitudes and relative distributions of pressures at the tie/ballast interface are important inputs for trackbed engineering design and analysis aspects. The pressures produced by millions of load applications ultimately affect the long-term performance of the track and the service lives of the component materials and layers. Ideally the interstitial pressure intensities at the tie/ballast interface can be reduced by distributing pressures uniformly over large contact areas thereby reducing abrasion, wear, and crushing of the bottom of the tie and the surface layer of the ballast. This also reduces the proportion of the loadings having to be supported by the ties and rail, lengthening the service life of the track.

Recent studies (1) indicated that a specially designed granular materials hydraulic pressure cell manufactured by Geokon was applicable in the laboratory for measuring trackbed pressures at the tie/ballast interface. This cell was developed for measuring pressures in materials containing large, sharp particles -- such as railroad ballast. Prior to the development of this thicker cell for ballast pressure measurements, thinner earth pressure cells, primarily designed for soil applications, but requiring cushioning layers for measuring ballast pressures, were used to satisfactorily measure pressures at the ballast/subballast/subgrade interfaces (2, 3, 4, 5).

The latter study (1) indicated that not only was the thicker cell applicable to reliably and consistently measure pressures, but varying the composition of the supporting ballast did not have significant effects on the accurate transmission of the pressure distribution. Test results using new ballast size material, worn ballast size material, and smaller top-size ballast material (more suitable for laboratory tests) were similar. However, the study assumed the cells would be placed below a solid tie, as had been the situation for the laboratory studies, when used for in-track applications.

This actually proved unsatisfactory (6) since it was impossible to consistently maintain firm contact between the bottom of the tie and cells embedded in the trackbed. On installation, cells were wedged tightly between the crosstie bottom and the open-graded ballast. During the passage of trains, however, impact loadings caused the cells to gradually migrate downward into the ballast. Therefore, the pressure readings decreased with time as a function of train traffic. In effect, the tie "bridged over" the cell.

Figure 2 shows two trackbed installations. The first, a modernly fouled wood tie track, was on the yard lead at TTI Railroad in Paris, KY. The other, near Flat Rock, KY, was on a Class 4 Norfolk Southern (NS) mainline having concrete ties and a thick ballast section with essentially negligible fouling.

Efforts to tighten the cells by inserting thin metal shims to fill the voids between the tie and ballast were only partially successful. The passage of trains still caused the cells to migrate downward, reducing measured pressure values. If the cells were over shimmed, the pressures for the initial several trains would be very high, though decreasing as train traffic accumulated.

Furthermore, the transducer housing containing the strain gages and the instrument cable had to be positioned within the crib area of the track. This negated the possibility of subsequently tamping and surfacing the track to consolidate ballast disturbed and loosened during cell installation.

SECTION 3 CURRENT STUDIES

The primary purpose of this laboratory study was to determine the applicability and repeatability of using recessed hydraulic granular materials pressure cells to accurately determine vertical

Figure 2. Two track sites used for initial attempts to accurately measure tie/ballast interfacial pressures, TTI Railroad yard lead (left) and NS mainline (right).

pressures at the tie/ballast interface in railroad track. The research simulated in-track conditions using a laboratory testing machine to load a section of rail and tie supported by specially designed ballast and subballast containment frames.

One research objective was to explore practices for positioning the cells without adversely affecting typical in-track pressure distributions. Specific focus areas included techniques for attaching the cells to the ties and the influence of cell location within the bottoms of the ties. These findings could ultimately be employed when inserting the cells into in-service track.

The researchers elected to recess the active portion of the cells, the housing containing the strain gages, and the instrument cable into the tie. This placed the active pressure cell face flush with the bottom of the tie, permitting direct contact with the ballast. Routing established the needed spaces in the tie for the pressure cell assemblies. Figure 3 shows views of a routed test tie and pressure cell. The experiment subsequently evaluated the effect of recessing the cells in the tie bottoms.

Figure 3. Routed tie section (left) and pressure cell (right).

SECTION 4 TEST TRACKBED

The prototype laboratory track and support simulated an in-service trackbed. This test section incorporated vertical resiliency so that a typical track deflection of 1/4 to 3/8 in. (6 to 9 mm) would be achieved during loading. The ballast containment box/frame provided lateral ballast confinement in a manner similar to an in-service trackbe. A special resilient, expandable containment frame contained the ballast, while providing minimal lateral support to unduly restrain the ballast from movement.

The subballast, ballast, wood tie, tie plate and rail were typical of materials used in revenue track. Figure 4 provides a typical view of the track and support during a test.

Figure 4. Track section and support used for laboratory tests.

4,1 *Ballast and Subballast*

Layers of granite ballast and limestone subballast provided support for the pressure cells. The ballast was a mixture of new and worn ballast. The worn condition was achieved by initially subjecting the ballast to the LA Abrasion test to partially round the aggregate particles and provide a larger percentage of fine-sized particles. The larger size particles (plus 1-1/2 in. (38 mm)) were removed to provide more uniform support. Care was taken to insure that the ballast and subballast layers in the prototype track section were adequately and uniformly compacted, with the top surfaces level.

The bottom compression platen of the testing machine was lengthened to 72 in. (1.83 m) using three aluminum beams. The 61 in. (1.55 m) long by 25 in. (0.64 m) wide subballast box frame was positioned on the beams. Nominal 2 in. by 4 in. (50 mm by 100 mm) wood framing studs were used for the subballast frame. The bottom of the frame was composed of 5/8 in. (16 mm) thick layer of plywood with a 3/16 in. (5 mm) thick rubber mat on the underside to simulate a typical resilient roadbed.

A slightly smaller size ballast frame was sized to rest upon the subballast and fit inside the subballast frame. The 2 by 4 wood frame was heightened using a resilient composite lawn edging material. This increased the ballast thickness under the tie, while providing minimal lateral support. The bottom was composed of 3/16 in. (5 mm) thick polyester fiber/rubber-backed floor carpet.

The ballast depth was increased an additional 5 in. (125 mm) using an inner rectangular frame composed entirely of edging material placed directly on the ballast layer. This provided minimal lateral support/resistance to the tie.

The total support for the tie thus consisted of 10 in. (250 mm) of granite ballast (modified #4A grade) and 4 in. (100 mm) of limestone subballast (locally known as dense graded aggregate and typically used for railroad subballast and highway pavement base material.)

4.2 Wood Tie Sections, Tie Plate, and Rail

The test employed short sections of a standard size 9 in. (225 mm) wide by 7 in. (175 mm) thick copper naphthenate treated wood tie. The tie section for single cell testing directly under the rail was 11 in. (275 mm) long.

A standard 8 in. (200 mm) wide by 14 in. (350 mm) long steel tie plate was positioned between the wood tie and rail. A 1/8 in. (3 mm) thick narrow spacer placed within rail seat negated the cant to ensure that vertical loads would be applied through the track and support.

The rail was a 10 in. (250 mm) long section of 136-lb rail conforming to AREMA specifications. This rail spanned the width of the tie plate.

SECTION 5 LABORATORY TEST EQUIPMENT AND DATA ACQUISITION

A typical construction materials Baldwin/Satec hydraulic universal testing machine, having a test range of 300,000 lbf (1,335 kN), applied static compression loads in 1,500 lbf (6.7 kN) increments to a maximum load of 6,000 lbf (26.9 kN). The range of loading on the pressure cells provided pressures exceeding 100 psi (690 kPa), exceeding typical loading magnitudes and pressures exerted on ballast by locomotives and loaded freight cars.

The Geokon Model 3515 granular materials pressure cells, are designed to measure dynamic pressure changes in granular materials like railroad ballast. The cell consists of two circular 8 in. (200 mm) diameter stainless steel plates welded together around the periphery, separated by a small gap (void) filled with hydraulic fluid.

The applied pressure squeezes the two plates together, thus creating fluid pressure between the plates. A pressure transducer in the cell's steel housing transforms the fluid pressure to a current signal. The transducer pressure range is 0 to 360 psi (0 to 2.5 MPa).

The recorded pressure is the average pressure on the active cell area of 50.3 in^2 (324 cm^2). The cell plates are sufficiently thick that they do not deflect locally under normal particle point loads. Thus, the contact forces from all aggregate particles bearing on the active area influence the pressure reading.

The test used National Instruments Model NI 9203 C Series Current Input Module data logger attached to a laptop computer. The 12Vdc module has eight analog -20 mA to 20 mA current input channels with a maximum 200 kHz sample rate. The tests employed a sampling rate of 2000 samples/s. Figure 5a shows a pressure cell and the data logging system.

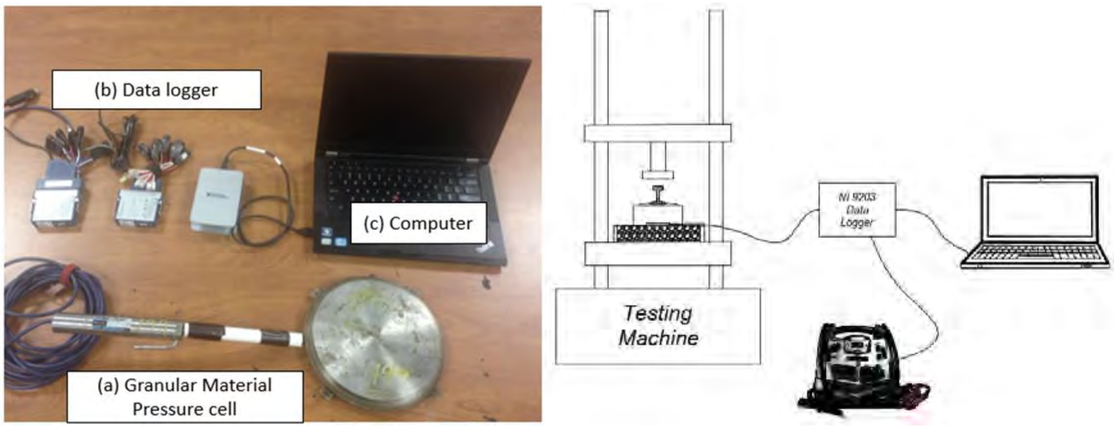


Figure 5a. Pressure cell and data logger module.

The team used LabVIEW to develop a user-friendly program to collect data. Figure 5b shows the main interface. The program can change units from mA to MPa and psi synchronously and show the pressure magnitude trace during the testing procedure. It also induces an automatic zero setting.

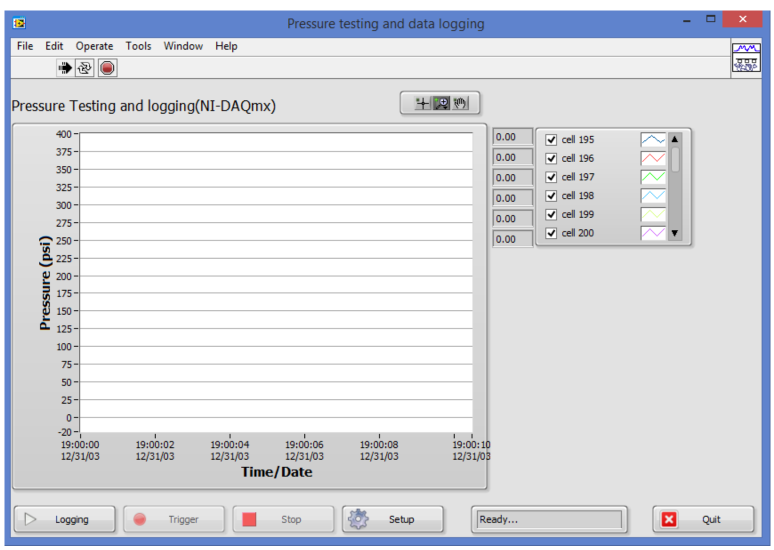


Figure 5b. Interface for LabVIEW program.

SECTION 6 SERIES OF LABORATORY TESTS

The team conducted several series of laboratory tests using conditions simulating in-track loading. A portion of these represented calibration-type validations given the known/controlled magnitude of the machine loading The area of load application from the tie to the ballast was the known active area of the pressure cell. By comparing the calculated machine pressure to the cell pressure indication it was possible to determine the cell accuracy. The validation tests confirmed that the cells did accurately measure the induced pressures.

Next, the effects of several variables were examined to help optimize installation practices used for placement of the cells within the trackbed.

The test sequence involved the following evaluations:

- Measurement Repeatability Within and Between Cells,
- Effect of Cell Location and Position, and
- Effect of Cell Attachment Procedures.

6.1 Measurement Repeatability Within and Between Individual Cells

Repeatability measurements employed four different cells. The only variable was the cell; loading magnitudes and test configurations remained the same. The loading from the tie to the ballast transferred through the complete area of the cell. The overhanging portion of the tie was not permitted to contact the ballast. The ballast was uniformly consolidated to minimize any variation during the test.

Six tests, involving cell pressure readings at four different loading magnitudes, were repeated for each of the four cells. The loading encompassed typical maximum magnitudes directly under a wheel load. The tie section was 11 in. (280 mm) long. A machine load of 6,000 lbf (26.9 kN) induced a pressure of 119 psi (822 kPa) through the cell on the ballast. This is about three times what is considered typical of pressures (neglecting impact) in a typical in-service trackbed.

Figure 6.1a illustrates the relationships of average machine induced stresses and the corresponding average cell measured stresses for the four cells. The results demonstrate near perfect correlation with the deterministic model line (R^2 of 0.9994).

Figure 6.1b shows the relationship for six repeated tests for individual cell # 88. The variations in cell reading are normally less than ± 2 psi (14 kPa) from the average for the repeat tests. The R^2 was 0.9997. The low variations between different cells were expected since each cell is individually calibrated. The low variations in repeat tests indicate that the laboratory test conditions are consistent and the cells are capable of accurately and consistently measuring tie/ballast interface pressures.

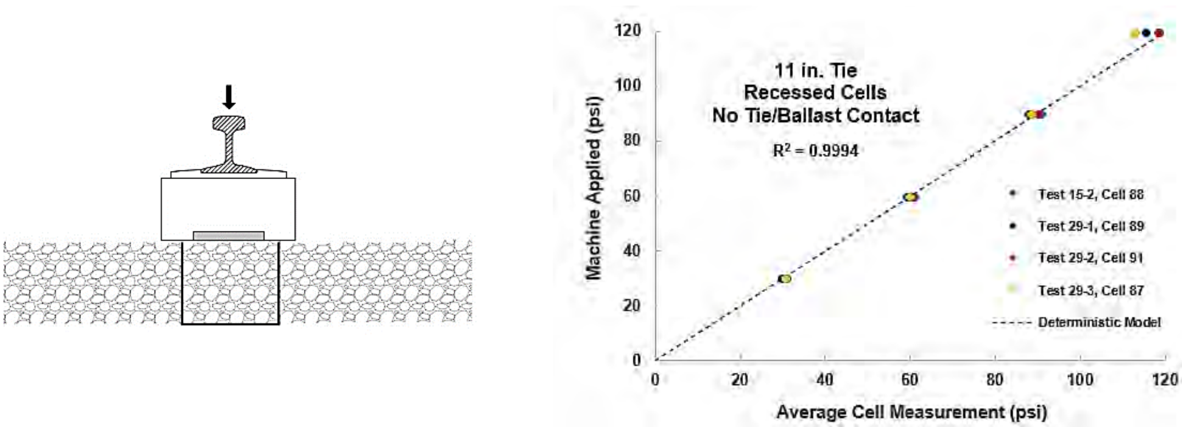
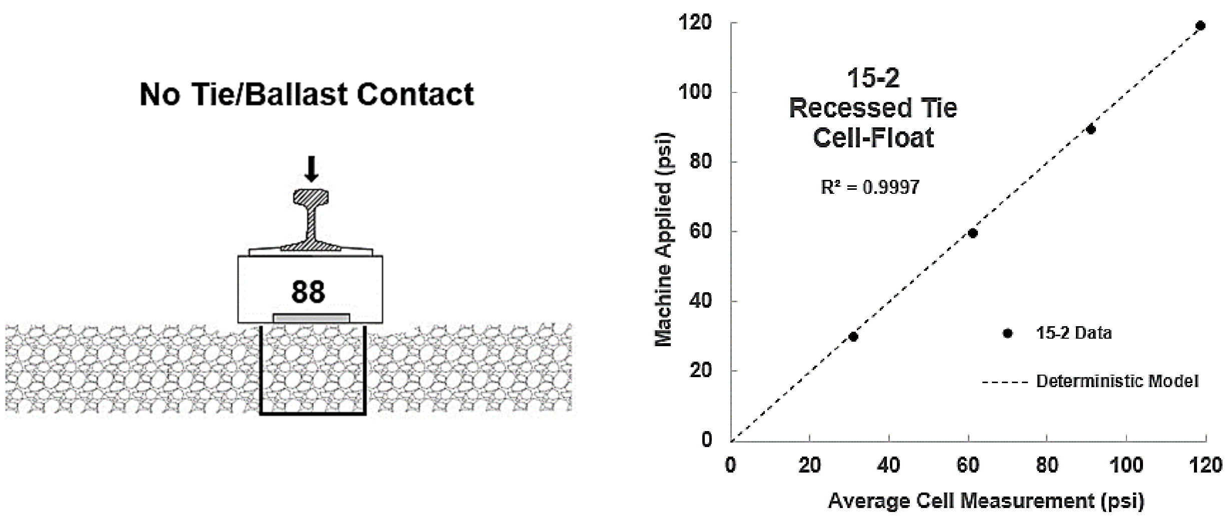


Figure 6.1a. Relationship between machine applied stresses and average cell measured stresses for four cells.



Machine		Test							
		(a)	*(b)*	*(c)*	*(d)*	*(e)*	*(f)*		
Applied (lbf)	**Applied (psi)**	**Measured (psi)**	**Measured (psi)**	**Measured (psi)**	**Measured (psi)**	**Measured (psi)**	**Measured (psi)**	**Range (psi)**	**AVG (psi)**
1500	29.8	32.8	31.5	31.1	30.8	30.7	30.8	30.7 – 32.8	31.1
3000	59.7	61.7	62.3	61.5	60.7	61.1	60.9	60.7 – 62.3	61.1
4500	89.5	92.6	92.2	91.8	89.9	90.2	90.1	89.9 – 92.6	90.0
6000	119.4	120.1	118.8	118.1	118.2	118.3	118.1	118.1 – 120.1	118.7

Figure 6.1b. Relationship between machine applied stresses and repeated cell measurement stresses for cell # 88; cell data similar for the other three cells.

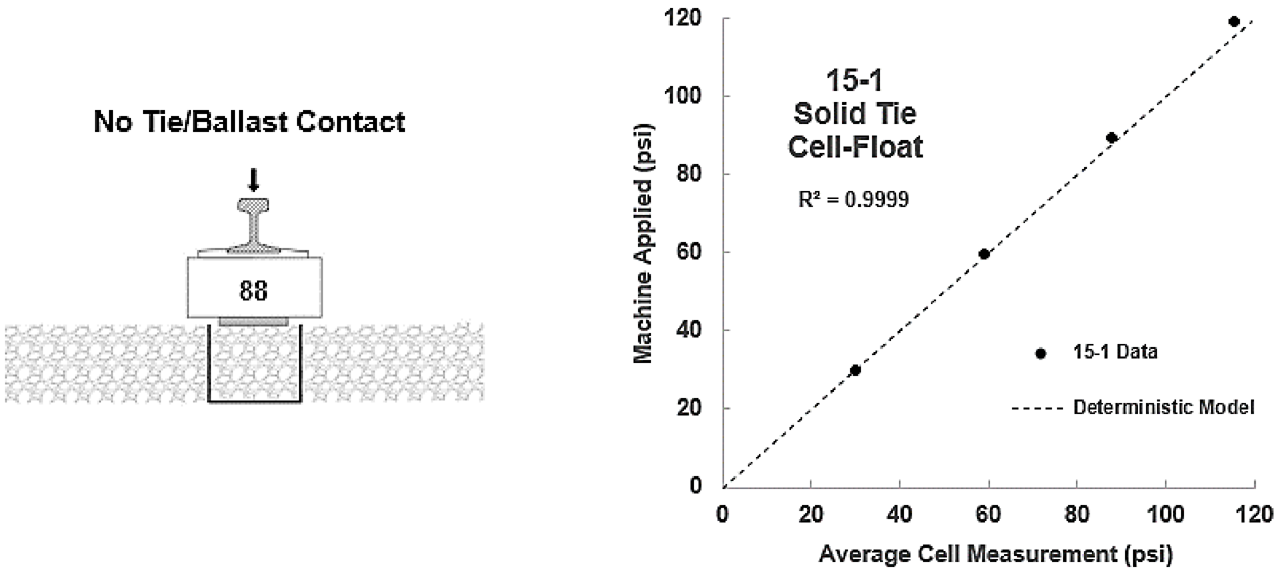
6.2 *Effect of Cell Location and Position*

Previous in-track tests using pressure cells placed below the tie at the tie/ballast interface have not yielded repeatable and consistent results. This research evaluated the effects of inserting the cells within a recessed cavity at the bottom surface of the tie. One-seventh of the thickness of the tie was removed (routed) to position the 1 in. (25 mm) thick cell and associated housing and cable. Figure 6.2a shows the routed tie and pressure cell.

Figure 6.2a. A typical routed tie with a recess and a pressure cell.

Repeated tests were conducted to assess the effects of recessing the cell within the tie. One setup employed a cell positioned directly below a solid tie; another recessed the cell within the bottom of the tie. Each setup underwent Six repeated tests at four different pressure levels. Only the cell surface contacted the ballast. Ballast was removed below the small area of the tie extruding beyond the cell surface to prevent a portion of the load from being carried by the tie. Figures 6.2b and 6.2c show the machine induced pressure and the cell pressure measurements.

The test data indicates very good correlation between the transmitted test machine pressure and the measured cell pressures. R^2s of 0.9999 and 0.9997 were achieved. The cell positioned below the solid tie recorded about 3 percent lower pressures for the two higher loadings. Test data variations between repeated tests were less than 1 psi (7 kPa). Therefore, recessing the cell within the underside of the tie, as compared to positioning the cell below the tie, has essentially no effect on transmitted pressures.

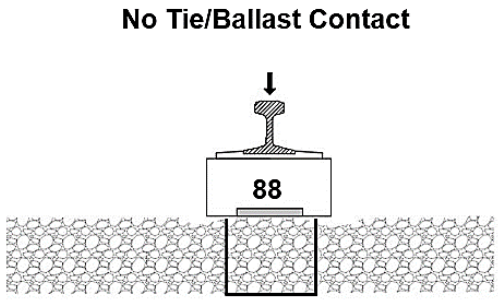


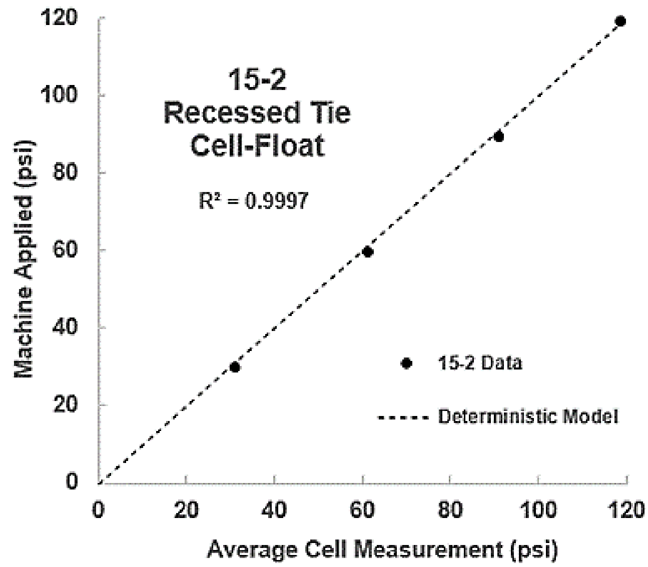
Machine		Test							
		(a)	*(b)*	*(c)*	*(d)*	*(e)*	*(f)*		
Applied (lbf)	**Applied (psi)**	**Measured (psi)**	**Measured (psi)**	**Measured (psi)**	**Measured (psi)**	**Measured (psi)**	**Measured (psi)**	**Range (psi)**	**AVG (psi)**
1500	29.8	30.1	29.5	30.2	30.5	29.8	30.2	29.5 – 30.5	30.0
3000	59.7	59.2	58.8	58.7	58.8	58.5	59.8	58.5 – 59.8	59.1
4500	89.5	86.8	87.1	87.6	87.5	87.1	87.8	86.8 – 87.8	87.6
6000	119.4	115.4	114.3	114.7	114.6	114.1	114.8	114.1 – 115.4	115.3

Figure 6.2b. Relationship between applied stresses and repeated cell measurement stresses for a solid tie with the cell positioned below the tie.

6.3 Effect of Cell Attachment Procedures

For the preceding laboratory test sequence, the cells below and within the tie were positioned and held secure by the test machine pressure pushing the ballast against the bottom of the cell. The cells were not secured directly to the ties, and could slightly adjust position. However, for in-track application it is desirable to fasten the cell to the tie This prevents the cell and tie from separating during installation. An attached cell cannot subsequently settle in the ballast so that the tie loading "bridges" across the cell, providing lower pressure readings than would be typical for non-instrumented ties.





Machine		Test							
		(a)	*(b)*	*(c)*	*(d)*	*(e)*	*(f)*		
Applied (lbf)	**Applied (psi)**	**Measured (psi)**	**Measured (psi)**	**Measured (psi)**	**Measured (psi)**	**Measured (psi)**	**Measured (psi)**	**Range (psi)**	**AVG (psi)**
1500	29.8	32.8	31.5	31.1	30.8	30.7	30.8	30.7 – 32.8	31.1
3000	59.7	61.7	62.3	61.5	60.7	61.1	60.9	60.7 – 62.3	61.1
4500	89.5	92.6	92.2	91.8	89.9	90.2	90.1	89.9 – 92.6	90.0
6000	119.4	120.1	118.8	118.1	118.2	118.3	118.1	118.1 – 120.1	118.7

Figure 6.2c. Relationship between applied stresses and repeated cell measurement stresses for a recessed tie and the cell positioned within the recess in the tie.

The researchers evaluated two methods for affixing the cells to the tie. One method employed screws though integral brackets on the cell body. A second involved retaining the cells using small flat metal corner braces screwed to the tie. The latter method partially demobilized the cells relying on the ballast to hold the cells within the recesses. Test results were compared to control tests without positive attachment (floating). Figure 6.3a shows the floating procedure and the two types of attachment procedures.

Figure 6.3b shows machine induced pressure versus measured cell pressure for the test. The test data indicates very good correlation between the transmitted test machine pressure and the measured cell pressures. All three procedures achieved R^2 values in excess of 0.999.

Figure 6.3a. The floating procedure and two types of attachment procedures.

Test data variations between the floating cell, screw, and corner brace attachments were less than 1 psi (7 kPa). Therefore, varying the cell attachment procedure has essentially no effect on altering transmitted pressures.

SECTION 7 CONCLUDING REMARKS

The research demonstrated a method for realistically calibrating and accurately measuring vertical applied pressures at the tie/ballast interface in the laboratory. It employed a simulated track section consisting of rail, tie plate, wood tie, ballast and subballast to measure pressures at the tie/ballast interface and to calibrate the cells.

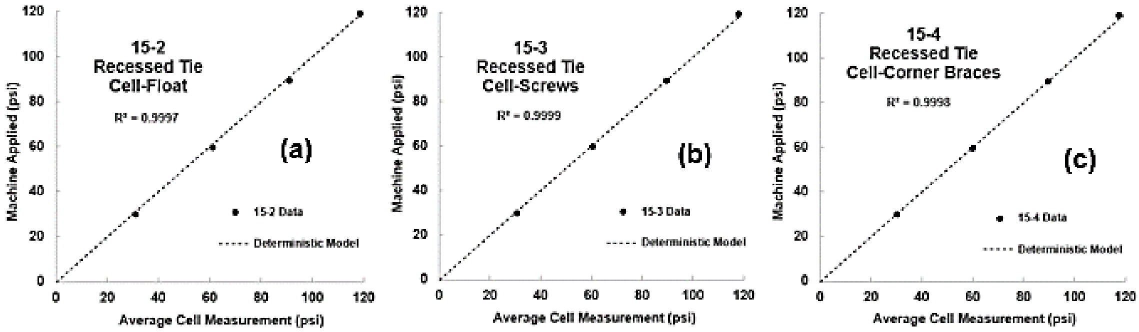
The trackbed support was designed to deflect, under typically in-track applied loading magnitudes, about 1/3 in. (8 mm); typical of in-track deflection for typical 36,000 lbf (161 kN) on-track wheel loadings.

The ballast containment frame was designed to minimally resist lateral movement of the ballast during load application, typical of an in-track trackbed. Particular attention was given to pre-compacting the subballast and ballast layers to simulate a seasoned track support system.

A granular material pressure cell, manufactured specifically to measure pressures of large-size graded aggregates, accurately measured vertical pressures under laboratory simulated in-track loading using known and controlled loadings. Near perfect correlation (R^2 of 0.9997) was obtained between repeated machine applied pressures and measured cell pressures.

The cells were recessed within the bottoms of the ties and protected from damage by the ballast, except for the active surfaces of the circular cells which were flush with the bottoms of the ties.

Test data indicated minimal variations between:



(a) Float

Machine		Test							
		(a)	*(b)*	*(c)*	*(d)*	*(e)*	*(f)*		
Applied (lbf)	Applied (psi)	Measured (psi)	Measured (psi)	Measured (psi)	Measured (psi)	Measured (psi)	Measured (psi)	Range (psi)	AVG (psi)
1500	29.8	32.8	31.5	31.1	30.8	30.7	30.8	30.7 – 32.8	31.1
3000	59.7	61.7	62.3	61.5	60.7	61.1	60.9	60.7 – 62.3	61.1
4500	89.5	92.6	92.2	91.8	89.9	90.2	90.1	89.9 – 92.6	90.0
6000	119.4	120.1	118.8	118.1	118.2	118.3	118.1	118.1 – 120.1	118.7

(b) Screws

Machine		Test							
		(a)	*(b)*	*(c)*	*(d)*	*(e)*	*(f)*		
Applied (lbf)	Applied (psi)	Measured (psi)	Measured (psi)	Measured (psi)	Measured (psi)	Measured (psi)	Measured (psi)	Range (psi)	AVG (psi)
1500	29.8	30.5	30.3	31.2	30.8	30.7	31.5	30.3 – 31.5	30.7
3000	59.7	59.8	60.1	61.3	60.9	60.5	60.8	59.8 – 61.3	60.4
4500	89.5	89.1	89.4	89.7	89.8	89.4	89.6	89.4 – 89.8	89.5
6000	119.4	117.2	117.8	118.1	118.2	117.8	117.7	117.2 – 118.2	118.0

(c) Corner Braces

Machine		Test							
		(a)	*(b)*	*(c)*	*(d)*	*(e)*	*(f)*		
Applied (lbf)	Applied (psi)	Measured (psi)	Measured (psi)	Measured (psi)	Measured (psi)	Measured (psi)	Measured (psi)	Range (psi)	AVG (psi)
1500	29.8	29.8	30.4	30.6	30.3	30.8	29.6	29.6 – 30.8	30.2
3000	59.7	59.5	60.1	60.7	59.8	60.3	60.1	59.5 – 60.7	60.0
4500	89.5	88.7	89.5	89.1	90.3	89.7	89.1	88.7 – 90.3	89.4
6000	119.4	116.8	117.3	117.2	117.6	117.1	117.4	116.8 – 117.6	117.5

Figure 6.3b. Relationship between applied stresses and repeated cell measurement stresses for recessed cells having different attachment procedures.

- Measurements using different cells for identical test conditions,
- Measurements for cells positioned below a solid tie (not practical for in-track applications) and cells recessed in the bottom side of the tie, and
- Measurements with cells "floating" loose within the recesses, cells attached with screws or cells attached with corner braces exhibited minimal variations.

For future in-track tie/ballast interface pressure measurements, the instrumented ties should be recessed by routing to provide space to embed and protect the active cell, transducer housing, and instrument cable.

The cells should be attached to the ties with either screws or corner braces arranged as depicted herein.

SECTION 8 ACKNOWLEDGMENTS

This research was primarily funded by the National University Rail Center (NURail), a U.S. DOT OST Tier 1 University Transportation Center and the Economic and National Science Foundation of China (51368021). Graduate Assistant Macy L. Purcell assisted significantly during the early phases of the research. Also appreciated are the financial assistance and cooperation of the Norfolk Southern Corporation.

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National University Rail Center - NURail
US DOT OST-R Tier 1 University Transportation Center

NURail Project ID: NURail 2016-UKY-R12d

In-Track Railway Track Tie/Ballast Interfacial Pressure Measurements Using Granular Material Pressure Cells

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TECHNICAL SUMMARY

Title

In-Track Railway Track Tie/Ballast Interfacial Pressure Measurements Using Granular Material Pressure Cells

Introduction

This report describes a research study to measure railroad track tie/ballast interfacial pressure using pressure cells specially designed for granular materials. Measurements were taken during a several month period at a test site located at Mascot, TN on a Norfolk Southern (NS) Railway mainline. The track has a 45 mph (72 km/h) speed limit and carries 37 million gross tons (33.6 million gross tonnes) annually. It is constructed with timber crossties and 136 RE continuous welded rail.

A group of six timber ties containing granular material pressure cells imbedded within recesses in the undersurface of the ties were successfully installed on an NS mainline track at Mascot, TN. A series of nine trackbed tie/ballast interfacial pressure measurements were taken of typical revenue trains over a seven-month period.

Approach and Methodology

The experiment employed new timber crossties routed to recess the pressure cell within the tie. Thus, the active surface of the pressure cell was flush with the tie bottom. Cabling was run through a recess to the tie end. This greatly reduced the likelihood of damage to the instrumentation during track surfacing and lining activity. The ties were installed such that multiple cells were directly under consecutive rail seats of one rail. Several ties also had cells either at the center or the rail seat of the opposite rail.

The report presents ballast pressure magnitudes and distributions and discusses results, including the effects of variable ballast support, wheel loadings, and flat wheels. Typical maximum vertical ballast pressure measurements directly under the rail seat, with ballast fully compacted, averages 20 psi (140 kPa) under the heaviest common revenue wheel loadings.

Findings

The consistency of recorded ballast pressures depended on the stability and tightness of the ballast support. The researchers expended considerable effort to provide consistent ballast conditions for the instrumented ties and adjacent, undisturbed (transition) ties. NS crews surface and tamped through and on either side of the test section. This, plus consolidation through normal accruing train traffic, resulted in consistent measurements through the section.

Ballast in the vicinity of the inserted instrumented recessed ties was disturbed (loosened) much less during the installation of the ties than previous attempts to excavate under existing solid ties to provide space for the cells positioned under the ties.

Imbedding the active area of the cells, transducer housing, and instrument cable within the recesses in the underside of the routed ties leaves the crib areas between the ties void of the cell instrumentation so that the instrumented ties can be adequately tamped to consolidate/compact the partially disturbed ballast.

Within cell pressure variations, for repeated train passages, are less variable than between cell variations for the same trains.

The average tie/ballast interfacial pressure under the rail seat for six-axle locomotives is 20 psi (140 kPa). The data is very consistent, with the maximum to minimum range by date averaging 18 to 22 psi (125 to 150 kPa) during the measurements for over twenty trains during the six-month period.

Conclusions

Initial testing revealed that the properties of the track and pressure distribution within the support are highly dependent on the relative consolidation/denseness of the ballast. Initial pressure measurements were considered marginally representative of typical pressure distributions for a well-consolidated, ballast supported trackbed.

The measured pressures are considerably lower than typically assumed for a high-tonnage ballasted timber tie track.

Pressures at the tie/ballast interface in the center of the track are initially very low. This is due to the center of the track not being routinely tamped during the installation of ties and surfacing track. However, with the passage of trains the track settles and the ballast in the center of the track readily contacts the ties providing the media for pressure transferal from the tie to the ballast.

Recommendations

Imbedding the cells within the ties and securing the cells to the ties negates the cells from settling in the ballast over time to develop gaps and “bridging”, however the ballast must be adequately and uniformly tamped to realize this advantage.

The ballast in the vicinity of the instrumented and approach ties should be uniformly consolidated/tamped to achieve equal vertical support for the ties assuring an equalized track modulus throughout the test area.

It is desirable for the test area to accumulate several months of normal train traffic and tonnage to further homogenize the trackbed support prior to drawing specific conclusions relative to the results of a testing program.

Publications

Rose, J.G. Clarke, D.B. Liu, L. and T.J. Watts. "Application of Granular Material Pressure Cells to Measure Railroad Track Tie/Ballast Interfacial Pressures". Transportation Research Record: Journal of the Transportation Research Board, 2018. 2671: In Press.

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SECTION 1 INTRODUCTION

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The consistency of recorded ballast pressures depended on the stability and tightness of the ballast support. The researchers expended considerable effort to provide consistent ballast conditions for the instrumented ties and adjacent, undisturbed (transition) ties. NS crews surface and tamped through and on either side of the test section. This, plus consolidation through normal accruing train traffic, resulted in consistent measurements through the section.

The report presents ballast pressure magnitudes and distributions and discusses results, including the effects of variable ballast support, wheel loadings, and flat wheels. Typical maximum vertical ballast pressure measurements directly under the rail seat, with ballast fully compacted, averages 20 psi (140 kPa) under the heaviest common revenue wheel loadings.

SECTION 2 BACKGROUND

The magnitudes and relative distributions of typical pressures induced by heavily-loaded revenue freight cars and locomotives at the tie-ballast interface have been difficult to quantify and assess. The American Railway Engineering and Maintenance-of-Way Association (AREMA) design methodology assumes that only the outer thirds of the timber tie conveys rail loads to the ballast. The center third of the tie is typically not tamped during tie installation and subsequent track surfacing. Thus, the ballast is not tightly consolidated in this area. As the tie is loaded through the rails, the center third of the tie is not initially in contact with the ballast, therefore no pressure distribution occurs.

Figure 2.1 contains the calculations for timber tie/ballast pressure determination as contained in the AREMA Manual for Railway Engineering (1). This procedure is largely based on the original Talbot design practice. The calculations are for a typical 100 ton (91 tonne) capacity car assuming

40 percent of the axle load is carried by the tie directly under the axle and further assuming an impact factor of 50 percent for a 50 mph (80 km/h) velocity.

The calculations indicate an average tie/ballast dynamic interfacial pressure of 65 psi (450 kPa) under the outer two-thirds of the tie. The analysis makes no allowance for variations in the transmitted pressure within the effective tie length. Were the center third of the tie to be in equal bearing, the average tie-ballast interface pressure decreases to 43 psi (300 kPa). However, this scenario has not been measured or confirmed from direct in-track tie-ballast interfacial pressure measurements.

$$\boxed{\mathbf{P_{Dyn} = P_S + \theta P_S}}$$

P_{Dyn} = Dynamic Wheel Load, lbf

P_S = Static Wheel Load, lbf

θ = Impact Factor

$$\boxed{\boldsymbol{\theta = \frac{33 \times V}{D_w \times 100}}}$$

V = Velocity, mph

$\mathbf{D_w}$ = Wheel Diameter, in.

$$\boxed{\mathbf{P_a = \frac{2P_{Dyn}}{\left(\frac{2}{3}\right) bL} \times \%}}$$

$\mathbf{P_a}$ = Tie/Ballast Pressure, psi

b = Width of Tie, in.

L = Length of Tie, in.

% = Percent of Wheel Load carried by Tie directly under the load, typically 40%

For a static 33,000 lbf wheel load, 33-in diameter wheel, 50 mph velocity, 9 in. wide and 8 1/2 ft long wood tie, the calculated tie/ballast interfacial pressure is 65 psi (450 kPa).

Figure 2.1. Typical calculations for tie/ballast contact pressures as contained in the AREMA Book of Recommended Practices (1).

SECTION 3 PREVIOUS TIE/BALLAST PRESSURE TESTING

Considerable effort has been expended at developing a consistent, reliable, and accurate procedure to measure pressures at the tie/ballast interface directly below the rail seat. This is the location considered to have the highest pressure. Following sections describe several initial tests, with the bulk of the paper addressing the most recent series of in-track tests at Mascot, TN.

3.1 Instrumentation

All tests employed Geokon Model 3515 granular materials pressure cells. These are designed to measure dynamic pressure changes in aggregate of similar size and grading to railroad ballast. A cell consists of two circular 8 in. (200 mm) diameter stainless steel plates welded together around the periphery, separated by a small gap (void) filled with hydraulic fluid. The pressure cells have an active area of 50.3 in^2 (324 cm^2). Applied pressure squeezes the two plates together, creating fluid pressure in the cell. The two plates are sufficiently thick so they do not deflect locally under the point loads from surrounding large aggregate particles.

A pressure transducer installed in the steel cell housing transforms the fluid pressure to a current signal. The measured pressure is the average pressure on the active area. The pressure range of the pressure transducer is 0 to 360 psi (0 to 2.5 MPa).

A National Instruments Model NI 9203 C Series Current Input Module was used for data acquisition. The 12Vdc module has eight analog -20 mA to 20 mA current input channels with a maximum 200 kHz sample rate. Figure 3.1.1 shows a pressure cell and the data logger module.

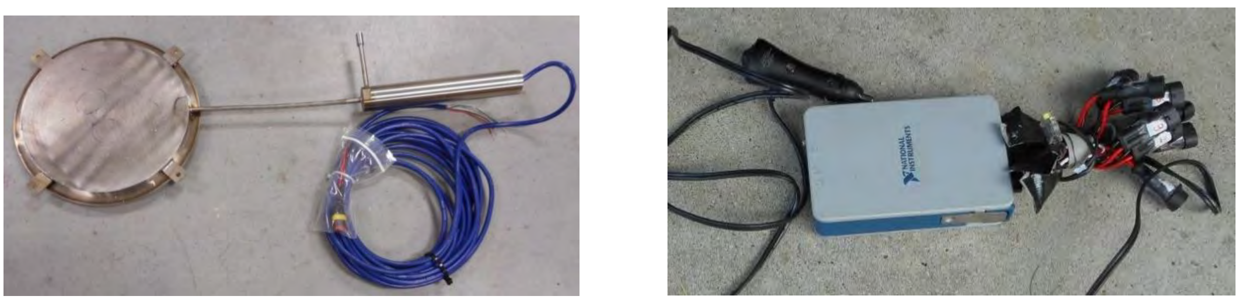


Figure 3.1.1. Pressure cell (left) and data logger module (right).

A user-friendly program for data collection was developed using LabVIEW. Figure 3.1.2 shows the main interface. The program can change units from mA to MPa and psi synchronously and show the pressure magnitude trace in real time during a test. It also induces an automatic zero setting. The experiment sampling rate was 2000 samples/s.

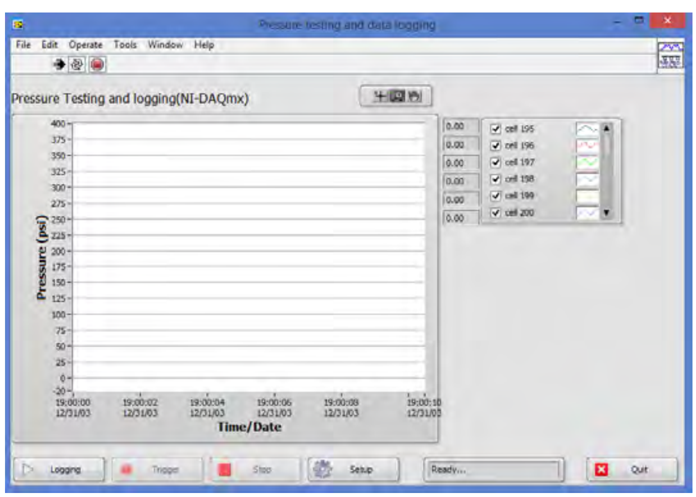


Figure 3.1.2. Interface for LabVIEW program.

3.2 *Laboratory Prototype Tests*

To develop and validate in-service test procedures, the researchers initially conducted laboratory tests of pressure cells installed in a prototype trackbed section consisting a rail, timber tie, ballast, and subballast. This simulated a typical trackbed.

A Model 3515 granular material pressure cell was installed in the tie section directly below the rail seat. Using a testing machine, the team applied controlled and known loadings to the rail/tie section. The cell measured the pressure at the tie-ballast interface.

The research determined that the cell measurements were accurate and consistent. Cell measurements correlated perfectly with calculated tie/ballast interface pressures based on applied loads (2,3). Measurement repeatability within and between individual cells was excellent for loadings encompassing typical maximum tie/ballast pressure magnitudes. The test data indicates that the cells accurately reflect pressure under laboratory conditions where the applied load can be measured and controlled.

3.3 *Transition to in-track installations*

The next step was to transition to in-track testing. In-service track differs from the laboratory tests in that a tie carries only a portion of the imposed wheel load. The continuous rail distributes the wheel load to several consecutive ties.

For in-track tests, the team placed pressure cells at the ballast interface below the rail seat for six consecutive ties. The experiment protocol was to maintain ballast in an undisturbed, compact condition when the cells were installed. However, this proved to be difficult in practice and ballast was disturbed.

Initial tests were conducted at two separate sites. A summary of these tests follows.

3.3.1 *TTI Railroad at Paris, KY*

The first site was on a TTI Railroad track connecting CSX Transportation's nearby mainline with the TTI Paris, KY yard. Maximum operating speed over the track was 10 mph (15 km/h). The lead track has 132 lb. jointed rail affixed with cut spikes to timber crossties. The trackbed was a tightly compacted, highly fouled mixture of ballast, degraded ballast, coal dust, and soil. The pressure cells were positioned at the ballast interface below the rail seat of in-situ ties, with the transducer housing and electrical cable in the crib area. Figure 3.3.1a shows the TTI track site.

Test results varied as a function of how tightly the particular cell was in contact with the bottom of the tie. Cells in direct contact with a tie recorded reasonably to extremely high pressure levels. However, cells that were positioned lower, thereby providing a "gap" or void between the tie and the cell, recorded pressures much lower, since the tie "bridged over" the cell.

Figure 3.3.1a. TTI Railroad test site (left) and Flatrock test site (right).

Some cells measured contact pressures in the 60 to 80 psi (415-550 kPa) range, considered typical for 4-axle weight locomotives. Other cells measured significantly lower pressures. The team shimmed adjacent cells measuring lower pressures with thin metal plates to fill the voids and provide uniform bearing.

Ultimately, longitudinal pressures for a typical 2-axle truck locomotive were distributed over 13 ties, with 38 percent of the axle load being carried by the tie directly under that particular axle. Figure 3.3.1b shows the load distribution pattern derived from the test data.

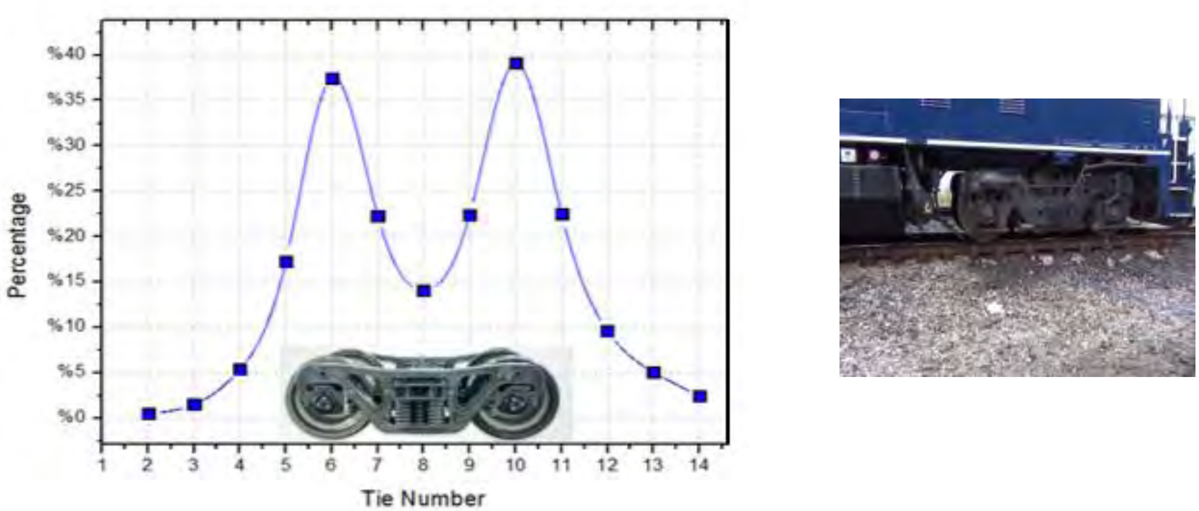


Figure 3.3.1b. Longitudinal distribution of the tie-ballast pressure for a 2-axle locomotive truck, axle spacing = 82 in. (2080 mm), wheel load = 32,000 lbf (142 kN).

Due to the extremely low tonnage and speeds and the fouled ballast, the cells were basically fixed in position so it was difficult to determine if the measured pressures were typical of those developed in an un-disturbed track. However, the relative longitudinal distribution of the pressures was evident from the pressures vs. time traces. Specific findings from the TTI tests follow:

- Pressure cells are applicable for measuring the interfacial pressures at the tie/ballast interface.
- The data acquisition system is adequate to develop realistic pressure vs. time traces.
- It is difficult to position the cells within fouled ballast to maintain constant contact with the underside of the ties. The cells are “peaked” or “bridged across” resulting in highly variable test data.
- Adequate consolidation/tamping of ballast under the instrumented ties and within the crib areas is essential to obtain accurate test results. However, this is impractical with the transducer housing and instrument cable within the crib area and the cell exposed under the tie,
- Cells must be positioned properly so that the applied pressures reflect undisturbed track.
- The tests produced results that can be used only for relative comparisons; absolute values are highly variable and not realistic for this test arrangement.

3.3.2 Norfolk Southern at Flatrock, KY

The second site was at Flatrock, KY on a very heavy tonnage Norfolk Southern (NS) Corporation mainline freight track. This was a concrete tie track positioned on clean thick ballast with asphalt underlayment support adjacent to a wheel impact load detector (WILD) installation. The maximum train speed was 45 mph (72 km/h). Figure 3.3.1a shows the Flatrock test site.

As with the Paris test, the cells were inserted at the tie/ballast interface under six consecutive ties of one rail. However, the disturbed ballast was reasonably loose in the vicinity of the cells, due to the effects of inserting the cells, so the ties initially “bridged across” the cells. Little to no force (pressure) from the axles passed through the cells. The team installed thin metal shims between the ties and cells so the ties would be in direct contact with the compacted ballast.

After multiple phases of shimming, each following two-weeks of train traffic, higher pressure magnitudes were initially obtained. The typical tie/ballast pressures obtained under locomotives or loaded freight cars varied significantly between cells. Excellent resolution of the pressure vs. time relationship traces were obtained showing the relative pressure as a function of axle loads (4).

Figure 3.3.2 contains the averages and ranges of pressure measurements at Flatrock over a period of time for the six cells. The magnitude of the pressure measurements was highly dependent on the position of the cells within the ballast. Ideally, the cells were in direct contact with the bottom of the ties and level with the adjoining ballast section. Furthermore, the cells should be fixed in the locations so as to maintain their positions along a level plane. Ultimate, the cells gradually settled in the ballast under continued train traffic and the “bridging” phenomenon reduced pressure readings.

Specific findings from the Flatrock tests follow:

- Ballast was disturbed (loosened) during the installation of the cells.

- There is no feasible method to adequately consolidate/compact the disturbed ballast without destroying the exposed instrumentation.
- Metal shims can be used to temporarily fill gaps between the underside of the ties and the surface of the cells, but do not provide long-term uniform support.
- It is impossible to position the cells on a uniform horizontal plane corresponding with the level of the undersides of the ties.
- Cells continue to settle in the ballast under train traffic with gaps developing between the underside of the ties and cells.
- Measured pressures vary significantly depending on the relative vertical position of the cells within the trackbed – can be "on a peak" thus higher pressures or "bridged over" thus lower pressures.
- The cells do accurately measure the pressures applied from the loadings at a given time based on contact conditions.

Clearly, the cell, transducer housing, and instrument cable need to be affixed within the tie bottom so that the tie can be tamped to uniformly consolidate the ballast without damaging the instrumentation.

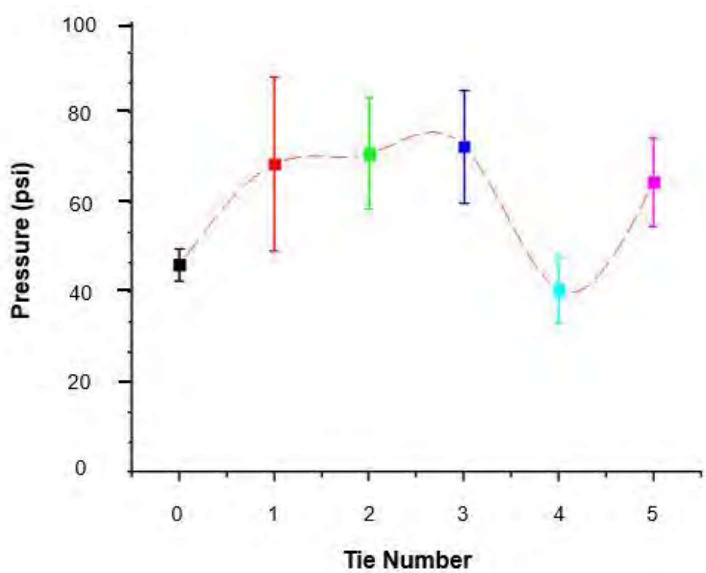


Figure 3.3.2. Longitudinal distribution pressures under consecutive ties, containing averages and ranges of data.

3.4 Lessons Learned for In-Track Installations

Based on the experiences at the Paris and Flatrock installations, future in-track test installations would have the pressure cells inserted in a routed (recessed) area in the tie with the cell's active surface flush with the tie bottom. This leaves the crib areas between the ties and ballast under the ties void of instrumentation so that the track can be surfaced and consolidated by tamping to obtain uniform ballast compaction. Attaching the cell to the tie negates the issue of a gap or void lowering applied pressures. The ability to tamp and consolidate the ballast uniformly should provide more consistent pressure distributions from tie-to-tie and from date-to-date.

SECTION 4 IN-TRACK TEST EQUIPMENT AND DATA ACQUISITION PRACTICES AT MASCOT SITE

The site subsequently selected to evaluate the applicability of encasing the cells in the underside of timber ties to accurately and consistently measure the pressures at the tie/ballast interface was on an NS mainline at Mascot, TN. The following sections describe recent activities associated with this test site and provide the major aspects of this paper.

The Mascot test site is located on a mainline track with 136 RE continuous welded rail secured with cut spike fasteners to timber ties. Ties are positioned on 20 in. (500 mm) centers and each tie is box anchored. The track support consists of standard NS mainline granite ballast on a well-seasoned roadbed. There are no indications of mud or fouling. NS personnel report that the area has a long record of stable roadbed/trackbed requiring minimal track maintenance. The test area was timbered and surfaced in November 2015.

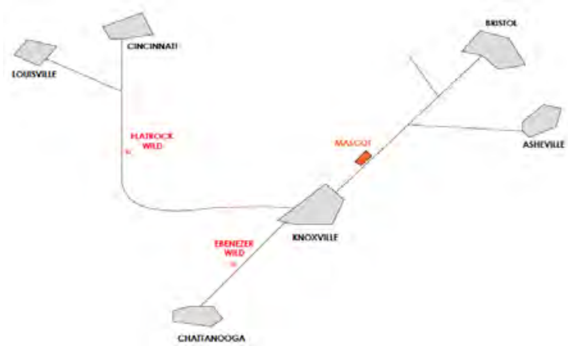
The site is on a horizontal tangent with a 0.25 percent vertical grade eastbound ascending. The track annually carries 37 million gross tons (33.6 million gross tonnes) of traffic, with a maximum train speed of 45 mph (72 km/h).

All east-west bound trains passing through Knoxville traverse the test section. A wayside automatic equipment identification (ADI) reader adjacent to the test site documents passing train consists. In addition, through trains pass over WILD sites west of Knoxville at either Ebenezer, TN or Flatrock, KY. Data from these installations permits subsequent comparisons of the tie/ballast pressures versus axle load.

Figure 4 contains a view of the test site, a schematic of the through NS mainlines in the Knoxville area, and a detailed schematic of the test site showing the locations of the instrumented ties.

4.1 Instrumented Tie Installation Procedures

To help obtain representative and consistent pressure data (4, 5), the tie bottoms were recessed to precisely conform to the shape of the cells. The recesses provided space for the active circular area of the pressure cell, the transducer housing, and the instrument cable. This operation was performed in a machine shop using a computer controlled router. Compliance with shop environmental requirements required copper naphthenate treated ties.



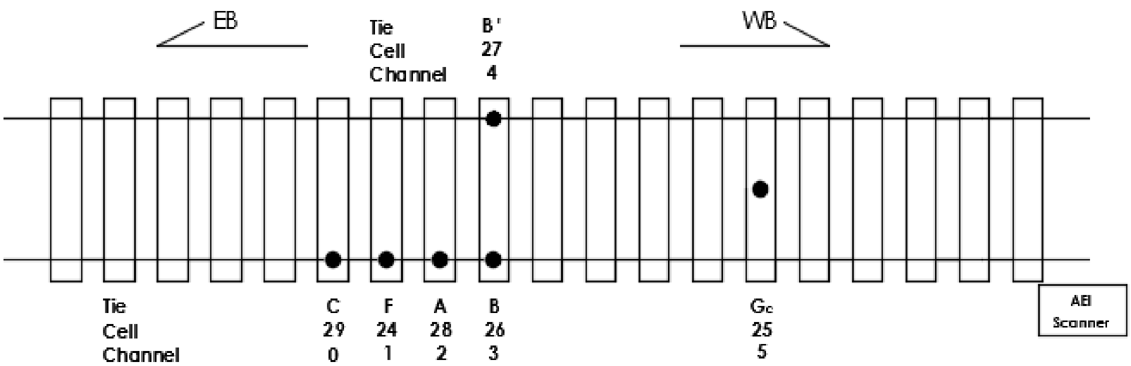


Figure 4. View of the test site, schematic of the NS through mainlines in the area, and detailed schematic for the test site.

The routing was precisely 1 in. (25 mm) deep. The active cell area was flush with the tie bottom to ensure representative ballast contact. The cell was attached to the tie using either screws or corner braces. Thin metal plates protected the transducer housing and instrument cable. A waterproof plastic box beyond the end of the tie contained the coiled cable length when not in use. With all of the instrumentation setup protected, railroad personnel could raise/surface the track using normal procedures.

Cells could be placed directly under one or both rails or the center of the tie. Figure 4.1.1 shows the three routing patterns, the routing process, and a cell partially installed within a tie prior to inserting the tie in the track.

NS provided the equipment and personnel to install the five instrumented ties. The crew took extreme care to avoid damage to the instrumented ties during the installation process. Figure 4.1.2 illustrates the handling and placement of the instrumented ties.

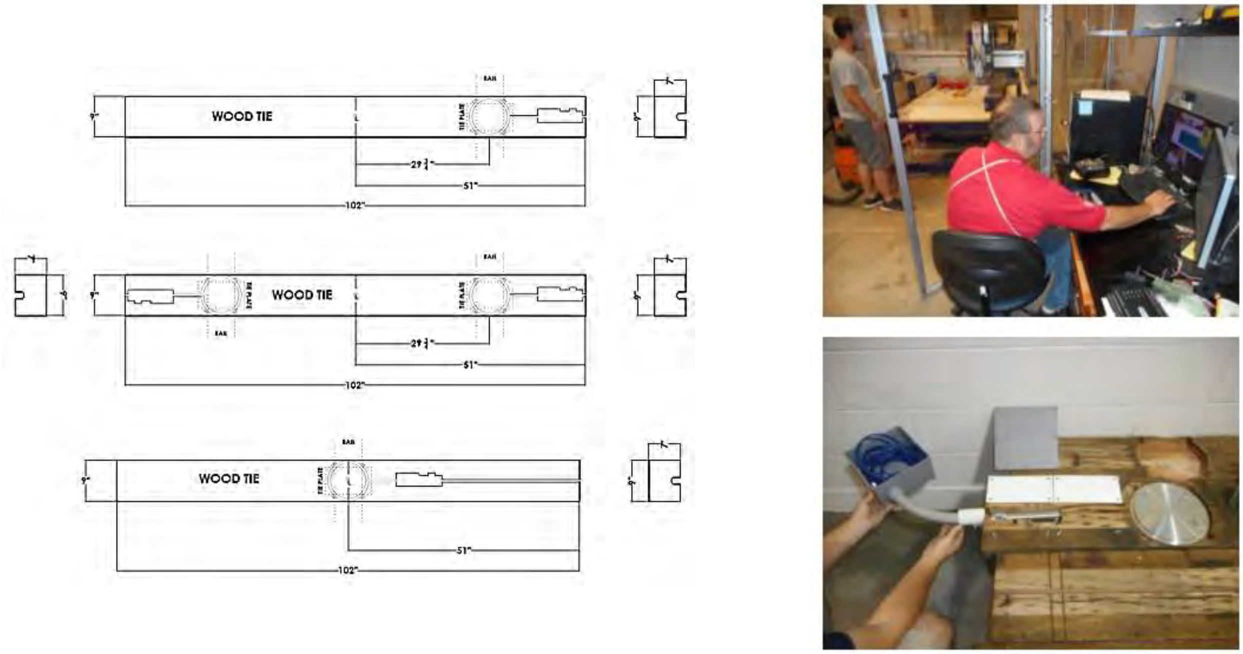


Figure 4.1.1. The three routing patterns for under the rails and in the center of the track, the routing process and a partially inserted cell and housing.

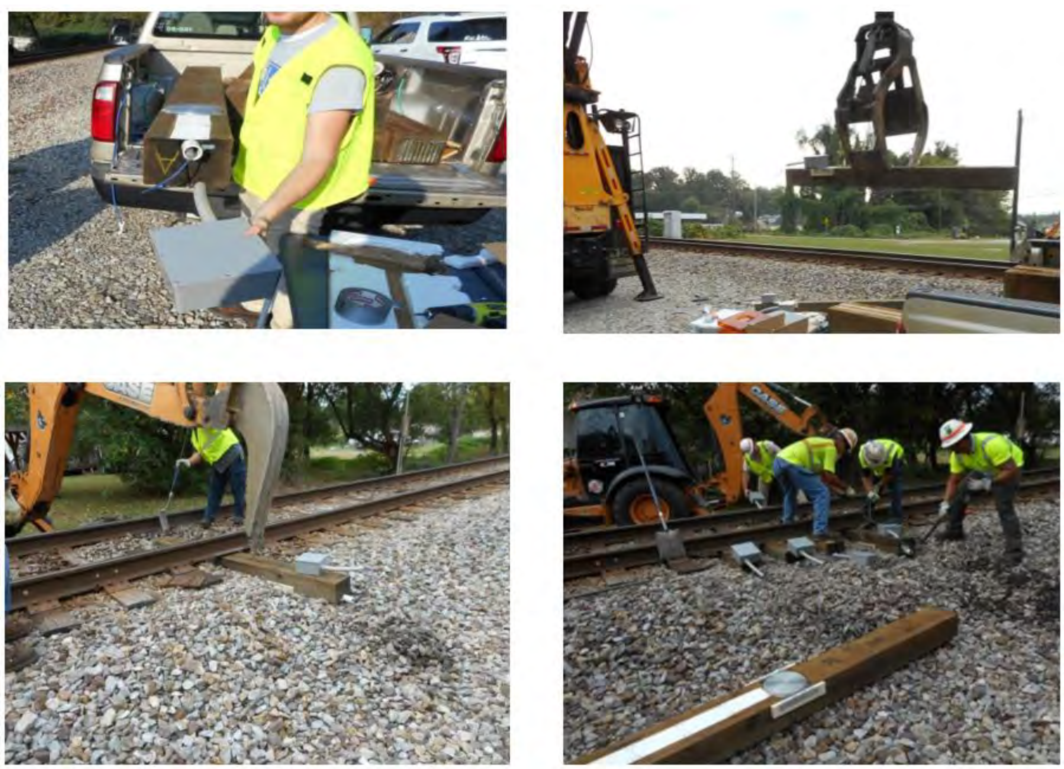


Figure 4.1.2. Inserting instrumented ties in the track.

The instrumented ties were immediately tamped and the initial testing procedure followed. Figure 4.1.3 shows the tamping process and the instrumented ties that were tamped. Initially only the instrumented ties were tamped, as shown in the upper sketch; subsequently after the initial test series the approach ties were also tamped, as shown in the lower sketch.

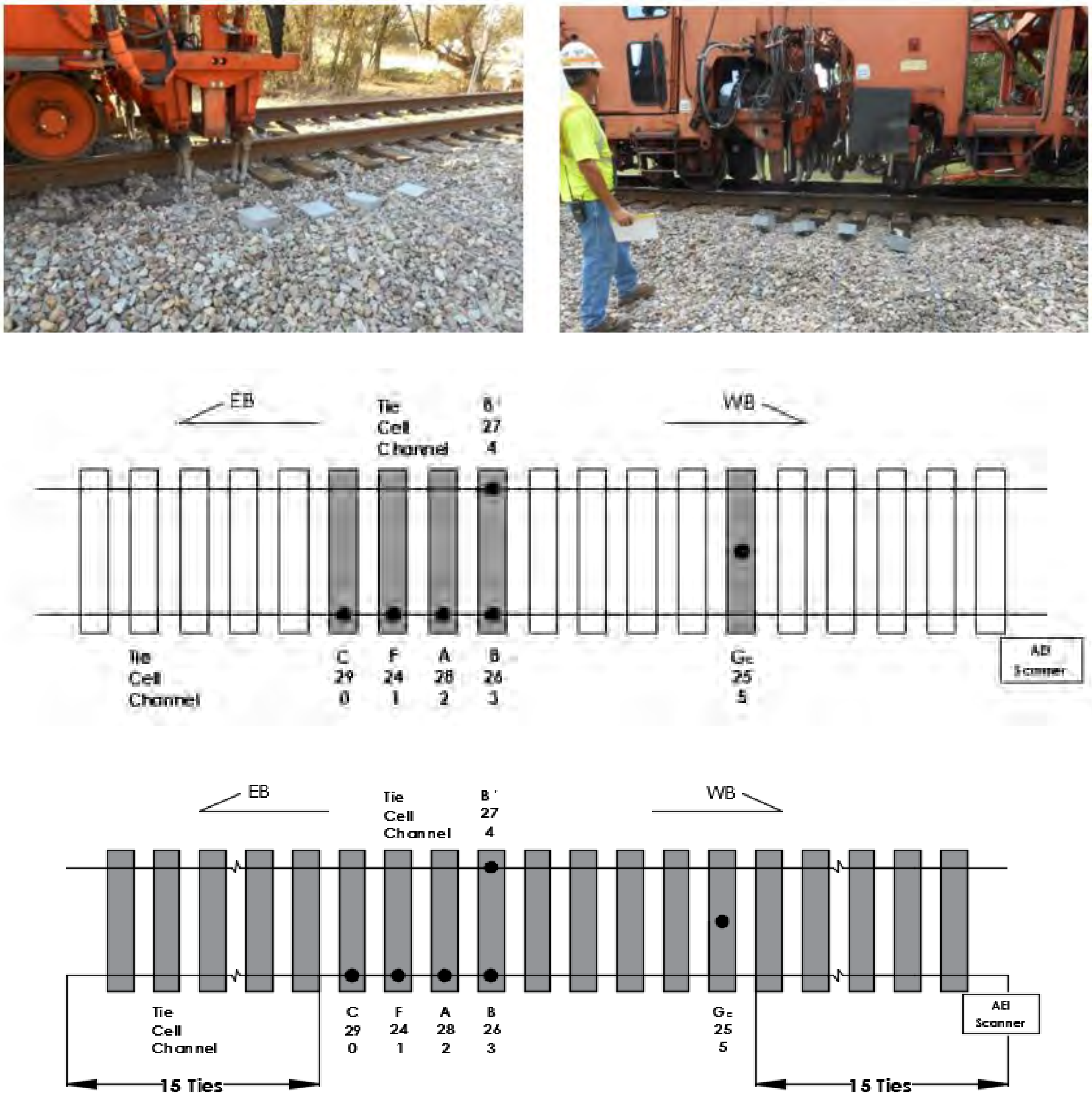


Figure 4.1.3. Track tamping process and the tamped instrumented ties shown in shaded gray. The upper sketch shows the instrumented ties (shaded) that were tamped for the initial two test series. After the initial two test series the 15 ties on each approach plus the ties in the instrumented area were tamped as shown in the lower sketch.

4.2 Test Procedure

Figure 4.2 contains a schematic of the data acquisition equipment and the wayside test equipment. The pressures exciting the six individual cells were simultaneously recorded in real time sequences (sampling rate of 2000 samples/s). The following information was obtained for each test train:

Train Number	Lead Locomotive Number
Time	Type of Train
Number/Axles of Locomotives	Direction of Travel
Speed of Train	Length of Train
Tonnage of Train	Number of Cars

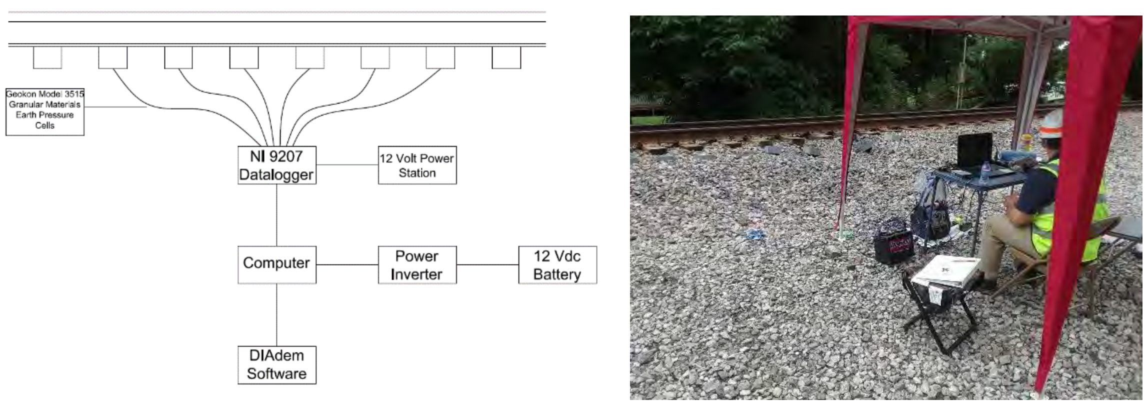


Figure 4.2. Schematic of data acquisition equipment and wayside measuring equipment.

Table 4.2 provides the chronologies for the tamping and testing sequences. For the initial two series of tests only the five instrumented ties were tamped per the layout in Figure 10. Additional tamping included nine ties in the instrumented area and fifteen ties on each approach. Later in the testing sequence, a more thorough double tamping procedure was performed to insure that the instrumented and approach ties were uniformly tamped so that the contact pressures would be equivalent for all ties. Since that time four more series of tests have been conducted, extending the test program to seven months.

4.3 Mascot Data Presentation

Nine series of pressure measurements have been conducted. Figure 4.3.1 contains typical pressure traces for a mixed freight train after all tamping was complete. The bottom trace is for

Table 4.2. Chronology for tamping and testing the instrumented ties.

Mascot, TN Site	
Date	
2016	
September 26	*Installed 6 Sensors (5 Ties)* *Single Tamped 5 Ties with Sensors* *Test – (variable & low)*
October 12	*Test – (variable & low)* *Single Tamped 39 Ties (15 + 9 + 15)*
October 26	*Test – (more consistent)*
November 7	*Test – (more consistent)*
November 15 & 16	*Double Tamped 39 Ties with Double Insertion*
November 28	*Test – (more consistent)*
December 15	*Test – (more consistent)*
2017	
April 13	*Test – (little change, except centerline of tie)*
April 27 & 28	*Test – (little change, except centerline of tie)*

the cell installed in the center of the track. All other cells were located under the rail seat per Figure 4.1.3. The train's consist was six 6-axle locomotives, shown by the 12 uniform peaks at the left of the traces, and twenty trailing freight cars (six loads and fourteen empties). Subsequent peaks of similar magnitude reflect axles of loaded cars.

Figure 4.3.2 shows the locomotives axles using an expanded time scale. Each individual peak is a wheel load, with six loads for each locomotive. Individual wheel loads for the locomotives are approximately 33,000 lbf (148 kN).

Table 4.3.1 provides average cell pressure readings for the nine test series. For simplicity of data presentation, only the locomotive pressure data is shown. The typical wheel loading from the 6-axle locomotives is approximately 33, 000 lbf (148 kN) so direct comparisons can be made from cell-to-cell and date-to-date. The pressure readings for the five cells directly under the rail are shown.

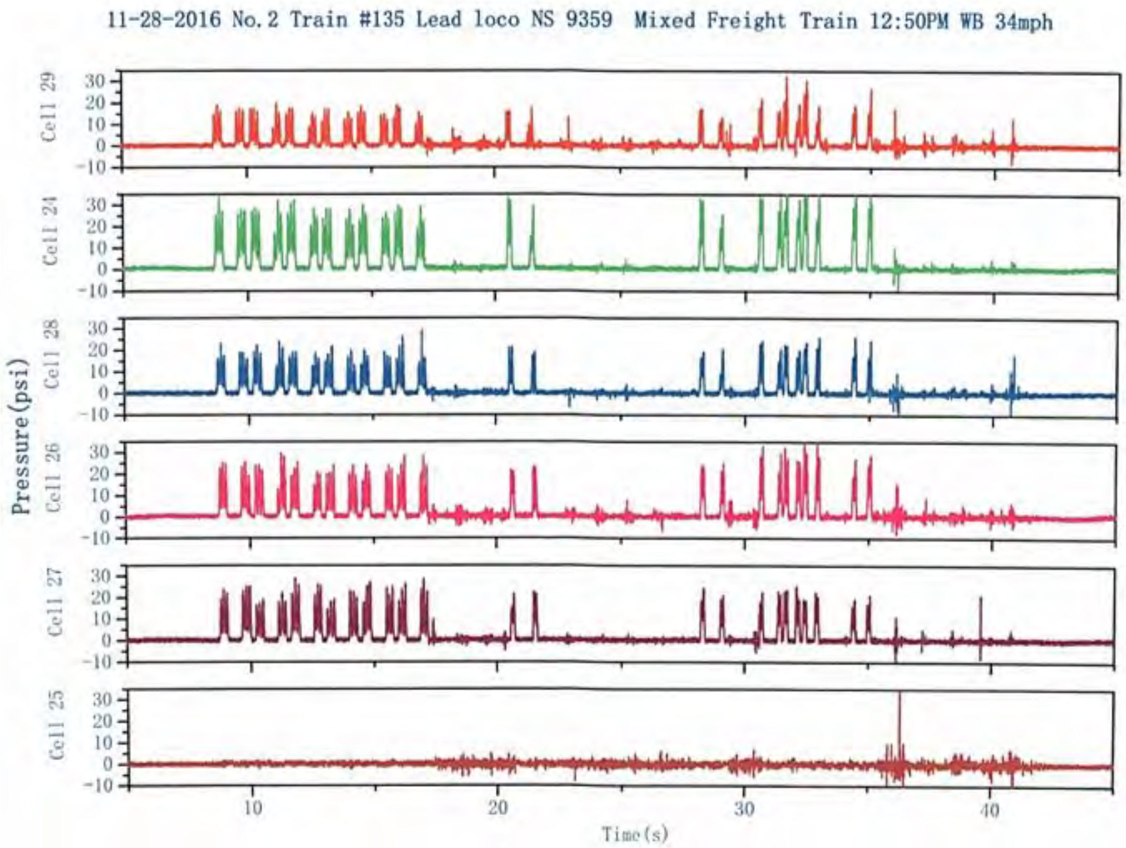


Figure 4.3.1. Pressure traces for a typical mixed freight train. The 12 peaks on the left represent the tie/ballast pressures recorded during the passage of the six locomotives, the remaining peaks represent six loaded freight cars. The upper five traces are for cells located under the rail, the bottom one is for a cell located in the center of the track.

The pressures were lower for the first two series of tests due to the ballast compaction being disturbed during insertion of the five instrumented ties. The instrumented ties "bridged over" the inadequately consolidated ballast. The pressures averaged just 20 psi (140 kPa), excluding the September 26 and October 12 series of tests.

Figure 4.3.3 graphically presents the average pressure data in Table 2, along with the range of pressure measurements, excluding data for the center-of-tie pressure cell.

Figure 4.3.4 summarizes the average pressure readings over time for the cell installed in the tie center. The pressures remained very low for the initial two months of traffic. As the tonnage accumulated during the seven-month period, pressures at the tie center increased to values near those recorded under the rail seat. The initial low pressures appear related to the lack of initial compaction/tamping of ballast under the tie middle third. Ballast is not in firm contact with the tie bottom. However, as the ballast compacts with accumulating tonnage, the tie settles and more load is transferred in the center third. Thus, the tie center cell records higher pressure reading.

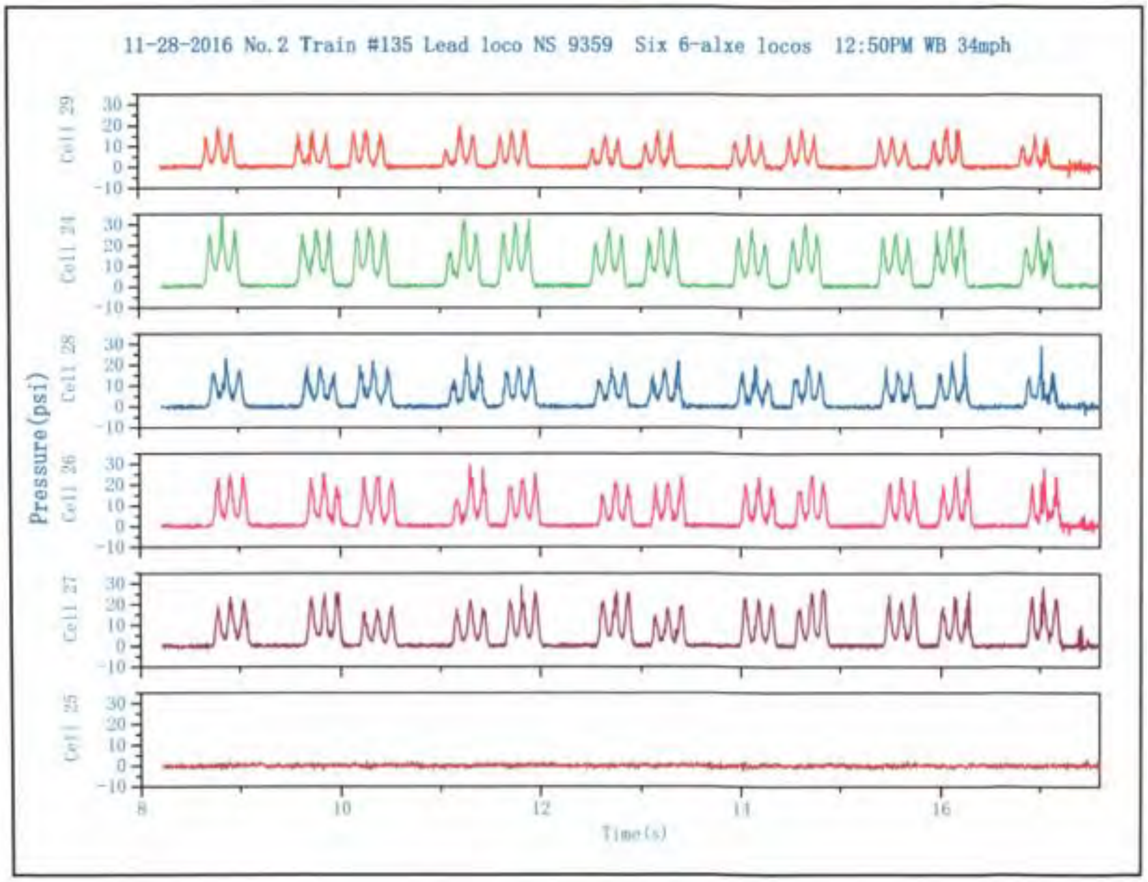


Figure 4.3.2. Expanded time scale for the six locomotives shown in Figure 4.3.1.

SECTION 5 CONCLUDING REMARKS

A group of six timber ties containing granular material pressure cells imbedded within recesses in the undersurface of the ties were successfully installed on an NS mainline track at Mascot, TN. A series of nine trackbed tie/ballast interfacial pressure measurements were taken of typical revenue trains over a seven-month period.

Ballast in the vicinity of the inserted instrumented recessed ties was disturbed (loosened) much less during the installation of the ties than previous attempts to excavate under existing solid ties to provide space for the cells positioned under the ties.

Imbedding the active area of the cells, transducer housing, and instrument cable within the recesses in the underside of the routed ties leaves the crib areas between the ties void of the cell instrumentation so that the instrumented ties can be adequately tamped to consolidate/compact the partially disturbed ballast.

Table 4.3.1. Average rail seat cell pressure, locomotives, by cell and test date.

Cell/Date	*29*	*24*	*28*	*26*	*27*	**Average**
9/26 [1]	**21.14**	**23.20**	**9.21**	**22.72**	**12.30**	17.71
10/12 [2]	**6.62**	**19.32**	**6.23**	**12.81**	**5.97**	10.19
10/26	**12.39**	**26.07**	**13.39**	**28.45**	**19.93**	20.05
11/7 [2]	**15.08**	**29.25**	**15.59**	**25.30**	**17.40**	20.52
11/28	**15.54**	**27.53**	**20.31**	**24.63**	**22.89**	22.18
12/15 [3]	**9.26**	**26.94**	**14.33**	**21.10**	**18.05**	17.94
4/13	**15.10**	**23.20**	**15.40**	**23.20**	**24.70**	20.32
4/27	**19.30**	**29.70**	**15.70**	**21.30**	**22.00**	21.60
4/28	**15.30**	**24.00**	**12.30**	**19.00**	**19.00**	17.92
Average	**14.41**	**25.47**	**13.61**	**22.06**	**18.03**	**18.71** [4]

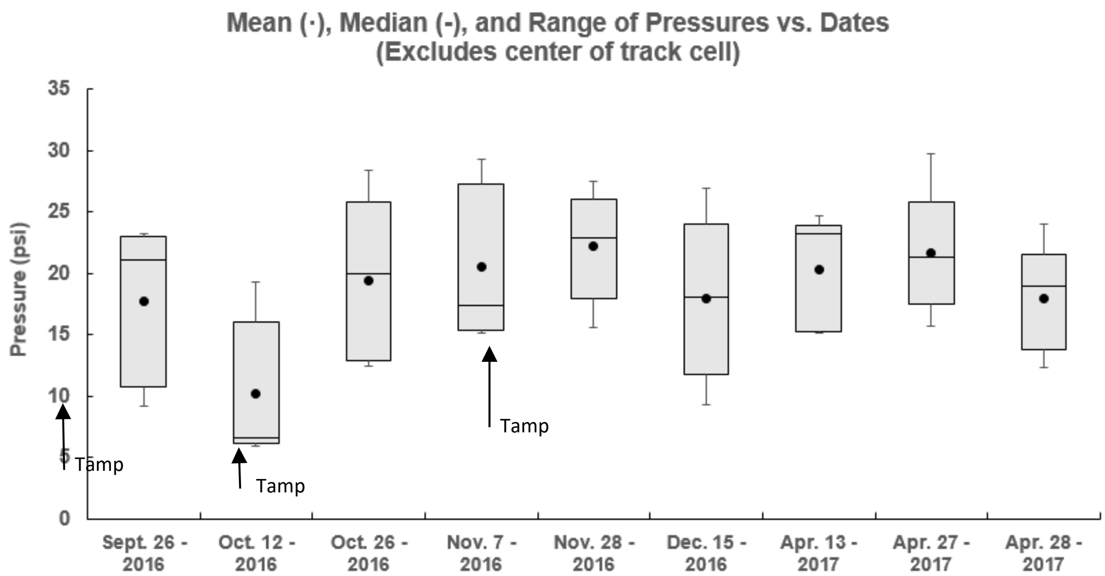
[1] Single tamping performed on instrumented ties
[2] Double tamping performed on instrumented and approach ties
[3] 12/15 data – Very cold temperatures
[4] Average is 20.08 psi excluding the 9/26 and 10/12 test series

Initial testing revealed that the properties of the track and pressure distribution within the support are highly dependent on the relative consolidation/denseness of the ballast. Initial pressure measurements were considered marginally representative of typical pressure distributions for a well-consolidated, ballast supported trackbed.

It is highly desirable to thoroughly tamp approach (undisturbed) existing ties in addition to instrumented ties to homogenize the compaction of the ballast in the vicinity of the ties.

Within cell pressure variations, for repeated train passages, are less variable than between cell variations for the same trains.

Imbedding the cells within the ties and securing the cells to the ties negates the cells from settling in the ballast over time to develop gaps and “bridging”, however the ballast must be adequately and uniformly tamped to realize this advantage.



<u>Note:</u> Initial tamping performed on 9/26. Follow-up tamping, including approaches performed on 10/12 & 11/28.

Figure 4.3.3. Graphical representation of the average pressures from Table 2 and the range in values.

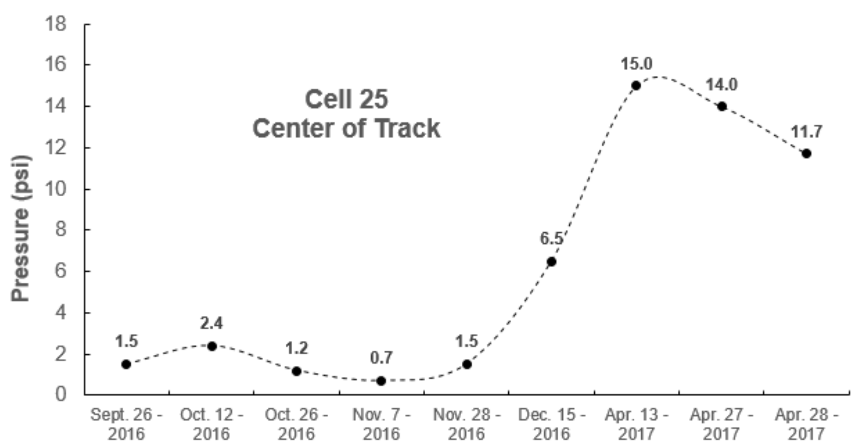


Figure 4.3.4. Average pressure readings for the cell installed in the center of the track for the nine test dates.

Disregarding the initial two series of measurements, during which the ballast appeared not to be adequately and uniformly compacted, the average tie/ballast interfacial pressure under the rail seat for six-axle locomotives is 20 psi (140 kPa). The data is very consistent, with the maximum to minimum range by date averaging 18 to 22 psi (125 to 150 kPa) during the measurements for over twenty trains during the six-month period.

The measured pressures are considerably lower than typically assumed for a high-tonnage ballasted timber tie track.

Pressures at the tie/ballast interface in the center of the track are initially very low. This is due to the center of the track not being routinely tamped during the installation of ties and surfacing track. However, with the passage of trains the track settles and the ballast in the center of the track readily contacts the ties providing the media for pressure transferal from the tie to the ballast.

The ballast in the vicinity of the instrumented and approach ties should be uniformly consolidated/tamped to achieve equal vertical support for the ties assuring an equalized track modulus throughout the test area.

It is desirable for the test area to accumulate several months of normal train traffic and tonnage to further homogenize the trackbed support prior to drawing specific conclusions relative to the results of a testing program.

SECTION 6 ACKNOWLEDGMENTS

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NURail Project ID: NURail 2016-UKY-R12e

Comparisons of Railway In-Track Tie/Ballast Interfacial Impact Pressure Measurements with Wheel/Rail Surface Impact Load Detector (WILD) Readings

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TECHNICAL SUMMARY

Title

Comparisons of Railway In-Track Tie/Ballast Interfacial Impact Pressure Measurements with Wheel/Rail Surface Impact Load Detector (WILD) Readings

Introduction

This report is based on a research study having the primary objective to compare wheel/rail surface impact loadings obtained from wheel impact load detectors (WILDs) with correspondingly transmitted trackbed tie/ballast impact pressures for revenue train operations. Particular attention was given to analyzing the effect of measured wheel/rail impact forces on transmitted tie/ballast interfacial pressures.

Approach and Methodology

A recently perfected procedure was used for measuring average pressures transmitted from the wheel to the track, directly under the rail contact patch, at the tie/ballast interface. The specially designed Geokon granular material cells were precisely positioned to measure the interfacial pressures along six consecutive ties, encompassing a complete wheel revolution. Data was obtained for a variety of freight trains over a period of several months during 2017. The test site is located at Mascot; just east of Knoxville, TN on a major east-west Norfolk Southern mainline.

A WILD is strategically located on each of the two mainlines diverging west of Knoxville. Pressure data was obtained on the combined line east of Knoxville for the eastbound trains that had recently traversed the WILD at either Flat Rock or Ebenezer, and for westbound trains that would soon traverse one of the WILDs.

Data obtained from the Mascot in-track tie/ballast interfacial pressure measurements, and data from the wheel/rail nominal and peak dynamic loadings obtained from the appropriate WILD installation for the corresponding freight trains, were compared to establish relationships and evaluate the effects of WILD impact loadings on transmitted trackbed tie/ballast interfacxial pressures for identical wheels.

Findings

This report describes the development of a method to measure average railroad track tie/ballast interfacial pressures using pressure cells specially designed for granular materials. The validity of the test method was initially verified with a series of laboratory tests using controlled loading magnitudes on prototype sections of trackbed with specially designed resilient frames/boxes for simulating typical in-track loading conditions. The selected procedure provided excellent correlation between controlled applied laboratory machine pressures and simultaneously measured cell pressures. The cells were recessed within the bottom of the ties for both the laboratory verification tests and subsequent in-track revenue train tests.

For the in-track tests, a series of cells were positioned under the rail at successive tie/ballast interfaces. Trackbed pressure measurements were conducted for numerous revenue freight trains during several months. After raising and surfacing the track, following installation of the instrumented ties, the track (mainly ballast) was permitted to further consolidate under normal accruing train traffic to insure that the ballast was tightly and uniformly compacted under the ties to effect transmission of equalized pressures from each of the ties to the ballast.

Measured pressures, directly below the rail/tie primary influence area at the tie/ballast interface, are considerably lower than trackbed pressures previously assumed for locomotives and loaded freight cars. These measured pressures range from 20 to 30 psi (140 to 210 kPa) for smooth wheels producing negligible impacts.

WILD wheel loading magnitudes obtained from nearby WILD wayside detectors were compared to trackbed pressure data for the same trains traversing the trackbed pressure cell test site. Various measured WILD parameters were compared to recorded trackbed pressures for loaded freight cars. The results indicate that increases in Peak wheel load values relate favorably to increases in recorded trackbed pressures. As an example, based on the regression relationship, as the Peak loadings increase -- the tie/ballast interfacial pressures increase by a factor of 2.25. Similar results were obtained comparing increased Nominal wheel loadings and increased tie/ballast interfacial pressures; pressures increased by a factor of 2.79.

The positive relationship between WILD loadings and corresponding tie/ballast interfacial pressure levels was further substantiated by comparing WILD Peak force and Nominal force relationships to measured tie/ballast pressures for an intermodal train. A R-squared value in the range of 0.8 was obtained for a variety of wheel loads.

The relationship between WILD Nominal wheel loading and tie/ballast pressure measurements is quite good for the higher magnitudes of wheel loadings. An R-squared value of 0.92 was obtained for a group of trains, excluding the light (empty car) wheel loads, and only considering the loaded car and locomotive wheel loads.

Conclusions

Measured pressures, directly below the rail/tie primary influence area at the tie/ballast interface, are considerably lower than trackbed pressures previously assumed for locomotives and loaded freight cars having smooth wheels producing negligible impacts.

Higher wheel load magnitudes and increased dynamic impacts due to imperfections in wheel-tread surfaces increases the magnitude of the pressures correspondingly transmitted to the trackbed support as indicated by increased in-track pressures measured at the tie/ballast interface. The added increase in tie/ballast pressure, due to the impact of imperfect wheel surfaces, basically serves in a similar manner as increasing the nominal permitted wheel loads. The resulting increased dynamic impact forces can contribute to higher degradation rates for the track component materials and more rapid degradation rates of the track geometry.

Recommendations

Based on the findings and conclusions for this study, following are several recommendations that should be implemented for further studies of this subject.

The stability and tightness of the ballast support influences the magnitudes and consistencies of the recorded ballast pressures. Considerable effort will be required to provide consistent ballast support conditions for the instrumented ties and adjacent undisturbed transition ties. Railroad maintenance crews must surface and tamp through the test section and adjacent approach ties. This effort along with normal accruing train traffic will subsequently result in reasonably consistent support pressure measurements throughout the test section.

Imbedding the cells within the ties and securing the cells to the ties negates the cells from settling in the ballast over time to develop gaps and "bridging", however the ballast must be adequately and uniformly tamped to realize this advantage.

The ballast in the vicinity of the instrumented and approach ties should be uniformly consolidated/tamped to achieve equal vertical support for the ties assuring an equalized track modulus throughout the test area.

It is desirable for the test area to accumulate several months of normal train traffic and tonnage to further homogenize the trackbed support prior to drawing specific conclusions relative to the results of a testing program.

Publications

Watts, T.J., Rose, J.G., and E.J. Russell. "Relationships between Wheel/Rail Surface Impact Loadings and Correspondingly Transmitted Tie/Ballast Impact Pressures for Revenue Train Operations". Proceedings of the 2018 Joint Rail Conference, Paper JRC 2018-6184, April, 2018, 10 pages.

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SECTION 1: INTRODUCTION

The undesirable effects of wheel/rail impact loadings on the track and supporting structure have been considered and evaluated for many years. A primary reason for the virtual demise of jointed rail track for mainline trackage was to eliminate the need of incorporating joints every 39 ft (11.9 m). The impact forces that ensued from the wheels having to cascade across the open section of the rail at the joint resulted in impact forces on the track and support structure with attendant settlement of the track in the vicinity of the joint. The individual rails became misaligned vertically across the joint and the wheels added additional impact forces and accelerated wear at the rail ends. This also increased impact forces and settlement of the track and support structure. Railroad track maintenance forces routinely raised and surfaced the track in the joint areas to reduce impact forces at the joints. This was particularly the prevailing situation when marginal quality trackbed support layers were often the norm, and when coupled with inadequate drainage, the trackbed provided less than desirable structural support.

Technological advances beginning in the mid-1900s to produce continuously welded rail (CWR) resulted in the advancement of installing rail without joints which was subsequently widely adopted as a standard for mainline track. A smoother and much improved ride quality ensued with greatly decreased impact forces at the wheel/rail interface. This in turn reduced track maintenance efforts and costs, which extended the life of the rail and track components.

Although the adoption of CWR for mainline, high-tonnage rail lines eliminated the primary source of wheel-rail impact forces, it alone did not completely eliminate impact forces. An additional source of impact forces was due to imperfections in the wheel tread contact surface as it rolled along the rail. These were typically flat spots, but also included imperfections in the steel, resulting in "rough" spots on the tread surface. The impact forces resulted in higher stresses in the wheel and the rail that could result in damage to the rail cars and lading and damage to the track and support structure.

The technology for continuously measuring contact forces, including normal and added impact forces due to wheel imperfections, was developed in 1983. Salient Systems (recently became a wholly owned subsidiary of LB Foster Company) was involved with the early development and applications of this technology. The incorporation of wayside wheel impact load detectors (WILDs) began in 1984 and by 1995 more than sixty systems had been installed in North America and Europe by Salient Systems.

The incorporation of WILDs is considered a standard practice for major railroads. These are strategically placed at selected locations throughout the system in order to routinely measure and evaluate the presence and severity of wheels producing high impact forces at the wheel/rail interface. Wheels having imperfections exceeding specified limits are detected, inspected, tracked, removed based on specified criteria, and replaced with new wheels based on industry standards.

SECTION 2: OBJECTIVES AND RESEARCH PLAN

The primary objective of this research was to compare wheel/rail surface impact loadings obtained from WILD defect detectors with correspondingly transmitted tie/ballast impact pressures for revenue train operations. Particular attention was given to analyzing the effect of measured wheel/rail impact forces on transmitted tie/ballast interfacial pressures.

A recently perfected procedure was used for measuring average pressures transmitted from the wheel to the track, directly under the rail contact patch, at the tie/ballast interface (1,2). The specially designed Geokon granular material cells were precisely positioned to measure the interfacial pressures along six consecutive ties, encompassing a complete wheel revolution. Data was obtained for a variety of freight trains over a period of several months during 2017. The test site is located at Mascot; just east of Knoxville, TN on a major east-west Norfolk Southern mainline.

A WILD is strategically located on each of the two mainlines diverging west of Knoxville. Pressure data was obtained on the combined line east of Knoxville for the eastbound trains that had recently traversed the WILD at either Flat Rock or Ebenezer, and for westbound trains that would soon traverse one of the WILDs. A view of the NS track layout in the area, with the test sites identified, is shown in Figure 2.

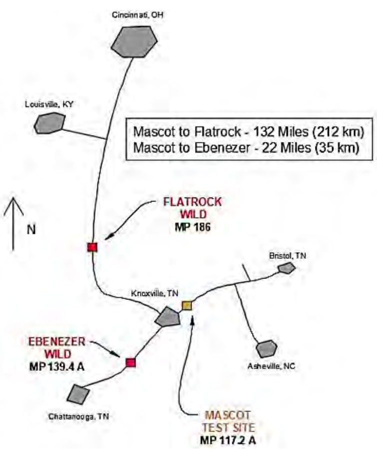


Figure 2. Track test site at Mascot and the locations of the Mascot test site and the two WILD test sites west of Knoxville for the main NS rail lines through Knoxville.

Data obtained from the Mascot in-track tie/ballast interfacial pressure measurements, and data from the wheel/rail nominal and peak dynamic loadings obtained from the appropriate WILD installation for the corresponding freight trains, were compared to establish relationships and evaluate the effects of WILD impact loadings on transmitted trackbed pressures for identical wheels.

SECTION 3: PRESSURE TESTING PROCEDURE

It has been desirable for years to develop a reasonably simple, accurate, and reliable method to directly measure average vertical pressure magnitudes and distributions at the tie/ballast interface in railroad trackbeds. Quantifying the magnitudes and relative distributions of pressures at the tie/ballast interface are important inputs for trackbed engineering design and analysis aspects. The pressures produced by millions of load applications ultimately affect the long-term performance of the track by reducing the service lives of the component materials and layers.

Ideally the interstitial pressure intensities at the tie/ballast interface can be reduced by distributing pressures uniformly over large contact areas, thereby reducing abrasion, wear, and crushing of the bottom of the tie and the surface layer of the ballast. This also reduces the proportion of the loadings having to be supported by the ties and rail, lengthening the service life of the track.

SECTION 4: PREVIOUS LABORATORY CONFIRMATION OF PRESSURE TESTING PROCEDURE

Recent research has shown that a specially designed Geokon Model 3515 granular material pressure cell is accurate and repeatable for measuring tie/ballast interfacial pressure for controlled loading conditions in the laboratory using simulated in-track loading conditions. (1).

A cell consists of two circular 8 in. (200 mm) diameter stainless steel plates welded together around the periphery, separated by a small gap (void) filled with hydraulic fluid. The pressure cells have an active area of 50.3 in2 (324 cm2). Applied pressure squeezes the two plates together, creating fluid pressure in the cell. The two plates are sufficiently thick so they do not deflect locally under the point loads from surrounding large aggregate particles.

A pressure transducer installed in the steel cell housing transforms the fluid pressure to a current signal. The measured pressure is the average pressure on the active area. The pressure range of the pressure transducer is 0 to 360 psi (0 to 2.5 MPa).

A National Instruments (NI) Model 9203 C Series Current Input Module was used for data acquisition. The 12Vdc module has eight analog -20 mA to 20 mA current input channels with a maximum 200 kHz sample rate. Figure 2 shows a pressure cell and the data logger module.

A user-friendly program for data collection was developed using LabVIEW. The program can change units from mA to MPa and psi synchronously and show the pressure magnitude trace in real time during a test. The experiment sampling rate was 2000 samples/s (Hz).

The tests showed that positioning the cell within a recessed (routed) portion of a wood tie flush with the bottom of the tie (Figure 4) did not affect the pressure distribution when compared to placing the cell below a solid tie. Recessing the cell is necessary for in-track measurements so that the cell, transducer housing, and instrument cable can be contained in the recess and be protected from track equipment routinely used to raise, surface, and tamp the ballast while adjusting the track geometry.

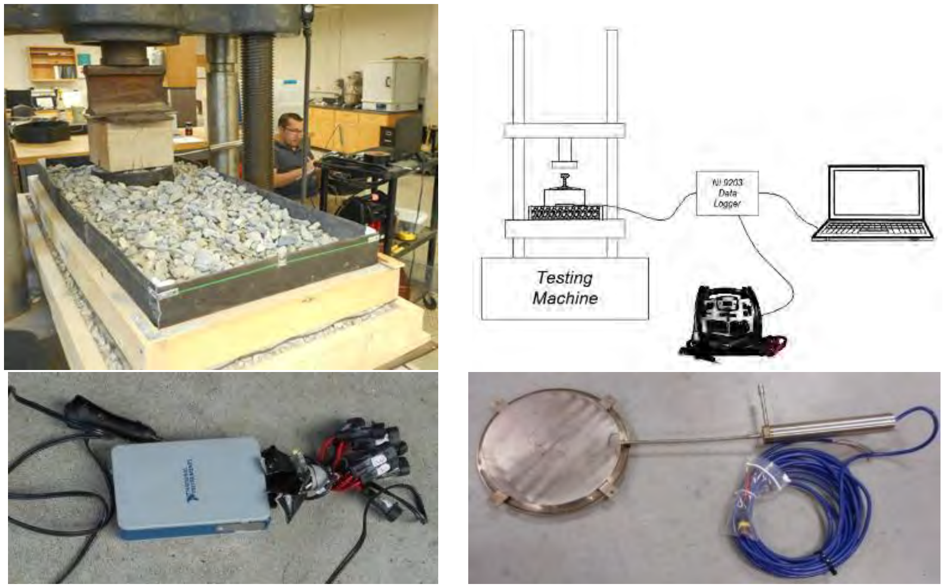


Figure 4. Laboratory trackbed support and loading system for simulated in-track loading conditions (top); pressure cell and data logger (bottom).

Furthermore, attaching the cell to the tie (necessary for in-track measurements) did not affect the transmitted pressures compared to just loosely inserting the cell. It is necessary for in-track measurements to attach the cell within the tie to prevent the cell from migrating downward in the ballast such that the tie actually “bridges over” the cell resulting in lower measured pressures than typical pressures (3).

SECTION 5: PREVIOUS IN-TRACK PRESSURE TESTING

The laboratory calibration and developed testing procedure confirmed that the Geokon Model 3515 granular materials pressure cell would be applicable for subsequent in-track tie/ballast pressure measurements provided the cell was imbedded within a recess in the bottom of the wood tie so that the active surface of the cell was even with the bottom surface of the tie. Furthermore, it was determined that if the cell had to be firmly attached to the tie.

In-track tie/ballast pressure testing has been underway at the Mascot test site for the past 18 months (2). Ten pressure cells were placed in the 1 in. (25 mm) routed portions of wood ties. NS maintenance crews removed the existing ties and installed the instrumented ties with minimum disturbance to the trackbed ballast, as shown in Figure 5a. Six of the cells are positioned under the rail for six consecutive ties on the north side of the track; test results for these six cells are used for the WILD data comparisons. Additional cells are positioned under the rail for two companion ties on the south side of the track. Also, cells are positioned under two ties in the center of the track to obtain comparative data. Schematics of the cell locations in the track at Mascot and the locations of the cell within the ties are shown in Figure 5b.

The Mascot test site is located on a mainline track with 136 RE continuous welded rail secured with cut spike fasteners to wood ties. Ties are positioned on 20 in. (500 mm) centers and each tie is box anchored. The track support consists of standard NS mainline granite ballast on a well-

Figure 5a. NS crews inserting instrumented ties in the trackbed at Mascot. The bottom view is an instrumented tie prior to installing in the track. The recessed cell will be on the bottom of the tie, level with the surface of the ballast.

seasoned roadbed. There are no indications of mud or fouling. NS personnel report that the area has a long record of stable roadbed/trackbed requiring minimal trackbed and roadbed maintenance. The test area was timbered and surfaced in November 2015. The quality of the ties in the test area is considered satisfactory for FRA Class 4 track.

The site is on a horizontal tangent with a 0.25 percent vertical grade eastbound ascending. The FRA Class 4 track annually carries 37 million gross tons (33.6 million gross tonnes) of traffic, with a maximum train speed of 45 mph (72 km/h).

All east-west bound trains passing through Knoxville traverse the test section. A wayside automatic equipment identification (ADI) reader adjacent to the test site documents passing train consists. In addition, through trains pass over a WILD site west of Knoxville, either at Ebenezer, TN or Flatrock, KY. Data from these installations permits subsequent comparisons of the tie/ballast pressures versus wheel loads at the Mascot test site.

Initial testing revealed that the properties of the track and pressure distribution within the support are highly dependent on the relative consolidation/denseness of the ballast (2). Initial pressure measurements were considered marginally representative of typical pressure distributions for a well-consolidated, ballast supported trackbed. It is highly desirable to tamp existing (undisturbed) ties on the approaches in addition to tamping instrumented ties to homogenize the compaction of the ballast in the vicinity of the ties.

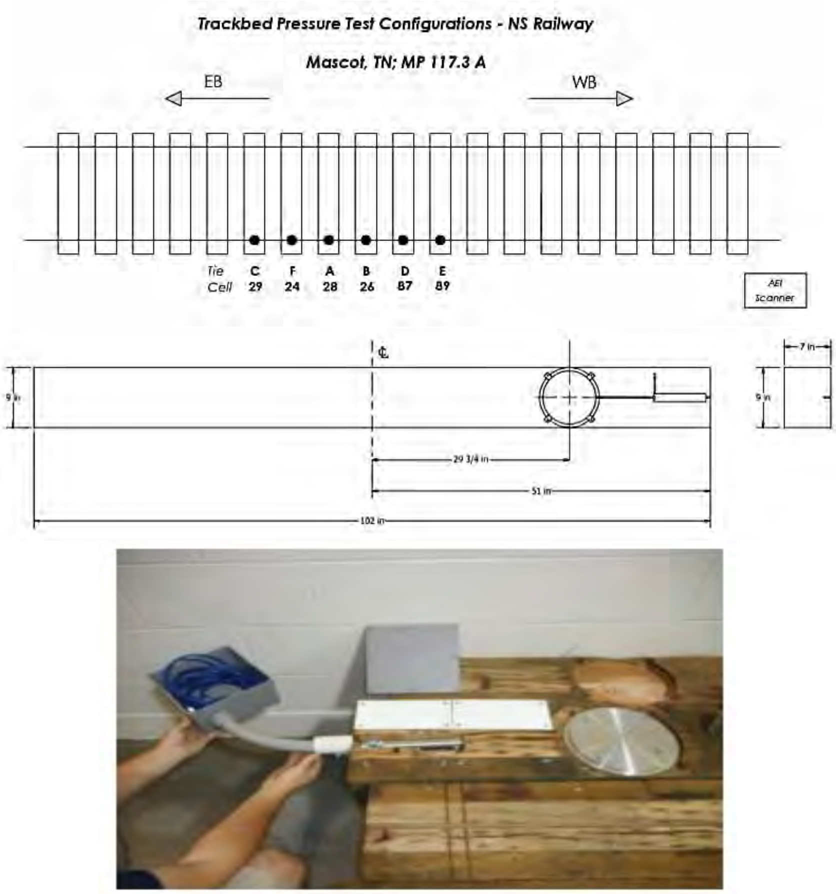


Figure 5b. Locations for the cells at the Mascot test site and locations of cells within the ties. The active portion of the recessed cell is even with the bottom surface of the tie and the remainder of the cell is protected from the ballast pressure and abrasion.

Subsequently a track raise was performed with additional tamping to insure that the level of compaction of the ballast was uniform for the approach ties and instrumented ties. This insures equalized vertical support for the ties so that an equalized track modulus will exist throughout the test area. Figure 5c contains views of raising, surfacing, and tamping the track and testing procedure for measuring tie/ballast interfacial pressures.

Cursory evaluations of the initial trackbed pressure data and comparable WILD data indicated that a relationship existed. This implied that wheels providing higher than nominal loading forces, thus impact, when traversing the WILD produced noticeable increases in tie/ballast pressures. However, several months of traffic provided sufficient compactive effort for the ballast to consolidate uniformly under the instrumented ties. Therefore, the data presented in the following sections was obtained for trains passing over the Mascot test site several months after the ballast was last manipulated.

Typical tie/ballast pressures for locomotives and loaded freight cars ranges from 20 to 30 psi (140 to 210 kPa) for smooth wheels producing negligible impacts. These measured pressures are considerably lower than typically assumed for a high-tonnage ballasted wood tie track (2).

However, the effect of increased wheel/rail impacts and peak loadings on the correspondingly transmitted pressures at the tie/ballast interface is significant, with increased pressures of several orders of magnitudes. This aspect will be discussed in detail in the following section.

Figure 5c. Experimental test equipment for obtaining pressure data (left) and raising, surfacing, and tamping process to uniformly consolidate the ballast throughout the test area and the experimental test equipment for obtaining pressure data (right).

SECTION 6: WILD TESTING PROCEDURE

A wheel impact load detector (WILD) consists of a series of individual strain gauges mounted on the neutral axis of web of the rail for a consecutive series of cribs for measuring vertical rail strain in order to calculate wheel loads. WILD sites are located on tangent track where lateral to vertical load ratios are typically less than 0.1. The track and support consists of premium size rail on concrete ties overlying a typical thickness of premium ballast supported by a well compacted thickness of subballast, typically hot-mix asphalt, and a well-compacted subgrade. This will reduce sources of variations within the track structure due to geometry and support conditions irregularities.

A WILD site normally involves about 200 to 250 ft (61 to 76 m) long section of track. This contains the track measurement zone, that is typically 50.5 ft (15.4m) long, and transitions on each ed. The rail is instrumented at various intervals to capture each single wheel's rotation at least two times. Peak loadings, which include impact, as well as nominal or average loadings are collected at 25 kHz frequency. The static wheel load is estimated by filtering the average or nominal forces from the peak forces by using an algorithm that analyzes variability along the site.

The PEAK wheel load is simply the highest recorded measurement from the strain gauge closest to the impact. It is the maximum impact force and is used for analyzing impacts for loaded cars and locomotives at a constant speed. For a given defect, the PEAK will tend to increase with vehicle weight and/or speed. The Association of American Railroads issues industry standards (criteria for repairs) for WILD alarms. The minimum alert threshold is 65 kips (290 kN).

The DYNAMIC Impact is the difference between the PEAK Load and the NOMINAL Load. This term is useful for analyzing intermediately loaded vehicles, but there are no industry threshold standards based on Dynamic Impact.

The PEAK Load divided by the NOMINAL Load is the RATIO or Impact Factor. It is useful for analyzing empty or lightly loaded vehicles. Although there is no alert threshold for Ratio, it is observed that once the ratio becomes higher than 3, it is likely that the vehicle will exceed the established Peak threshold when heavily loaded. These relationships are shown in Figure 6a.

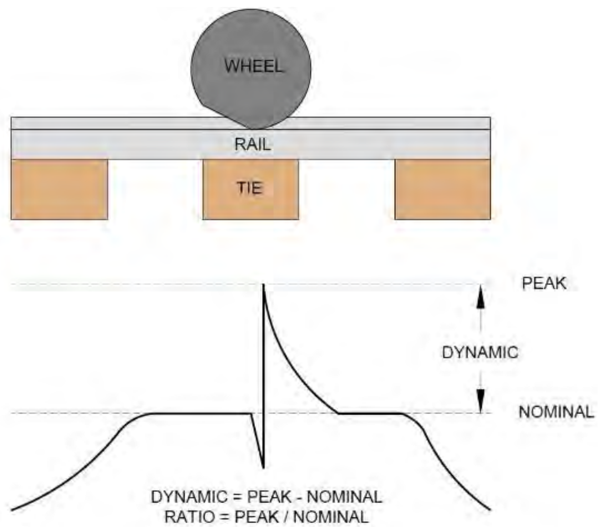


Figure 6a. WILD Measurements and Relationships for Peak Force, Nominal Force, Dynamic Impact, and Impact Ratio.

The Salient WILD design, being the initial type developed in the 1980s, is the most widely used system in the world today, with more than 200 installed worldwide to date. Over 90% of the WILD systems in the U.S. are Salient products. These evaluate millions of wheels per day throughout the international railway systems that detect and alarm when excessive wheel vertical impacts occur, so that the defective wheels are identified for inspection, tracking, treatment, and subsequent removal as standards dictate. A view of a Salient Mk-III WILD used to gather data for this study is shown in Figure 6b. This represents one of NS's fifteen WILDs designed and installed by Salient Systems.

Figure 6b. Typical Salient Mk-III WILD Installation.

The instrumented zone for the measurement of vertical forces exerted by each wheel of a passing train consists of a series of strain gage load circuits, micro-welded directly to the neutral axis of the rail.   Signal processors, housed in a nearby enclosure, analyze the data to isolate wheel tread irregularities. If any wheel generates a force that exceeds a customer-configured alarming threshold, a report identifies that wheel for subsequent action. Depending on operating procedures, multiple alarm thresholds can be configured. The reports are distributed in real-time to interested parties such as rail traffic control centers and vehicle repair shops.

WILDs are considered a strategic device for the protection of rail infrastructure. High impacting wheels can dissipate on the order of 25 horsepower each, degrading track, ballast and bridge structures, while reducing bearing and other vehicle component lives. Over time, the repetitive load cycles of defective wheels may result in rail fractures.

Particular attention was given in this research to determining the relationship of the relative increase in ballast pressures at the identified imperfect wheel traversed at close by WILD site that measured the magnitude of the increased vertical dynamic force.

SECTION 7: DATA PROCESSING

In its standard form, WILD data is produced in an axle domain with loads in its corresponding range. Being in this form allows for maintenance crews to directly identify wheel defects, which can be addressed downstream from the sensor. Although this a helpful format for maintenance purposes, the pressure measurements recorded at the tie/ballast interface in the track are in a frequency domain, which makes comparisons between the two somewhat difficult. Additionally, these WILD reports are based on a format that describes cars in an A or B-end category, and re-orients them for the convenience of engineering and maintenance crews. As additional processing for this study, individuals from Norfolk Southern Railway and LB Foster (Salient Systems) assisted in re-formatting the WILD data into a “victim’s” perceptive, which permitted matching wheel-for-wheel on the correct side of the test track.

In order to show the relationship between tie/ballast interface pressures and WILD force measurements, the pressure data recorded also had to be processed into an axle domain rather than a real-time/frequency domain in its raw existence. This was initially performed using a waveform peak operation in a software package called DIAdem, produced by National Instruments. Each peak, at least in a smooth wheel condition, is the corresponding force exerted from each axle. Although the operation is simple to use, the output file still identifies several irregular spikes and wave signatures that must be addressed for quality control. After manually checking each representative axle pressure, most irregularities are taken care of by taking an average over the irregular wave signature. After the data is processed, giving a pressure reading for each corresponding axle, several relationships can be plotted and analyzed with finer detail.

Using data provided by LB Foster, serval parameters can be analyzed to identify a tie/ballast interface pressure response relationship on several trains measured from June to November of 2017. Those parameters are Nominal Axle Loads (KIPS), Axle Load Peaks (KIPS), Dynamic Impacts (Peak - Nominal), and Impact Factor Ratios (Peak/Nominal). A schematic of the relationship between all of these parameters was depicted in Figure 6a.

SECTION 8: TYPICAL TRACKBED PRESSURE VALUES AND TRACES

Figure 8a contains plots in real time domain of trackbed pressures recorded by six cells, positioned directly under the rail at the tie/ballast interface, during the passage of a mixed freight train containing both empty and loaded freight cars. The forward half of the train, including locomotives, is exhibited.

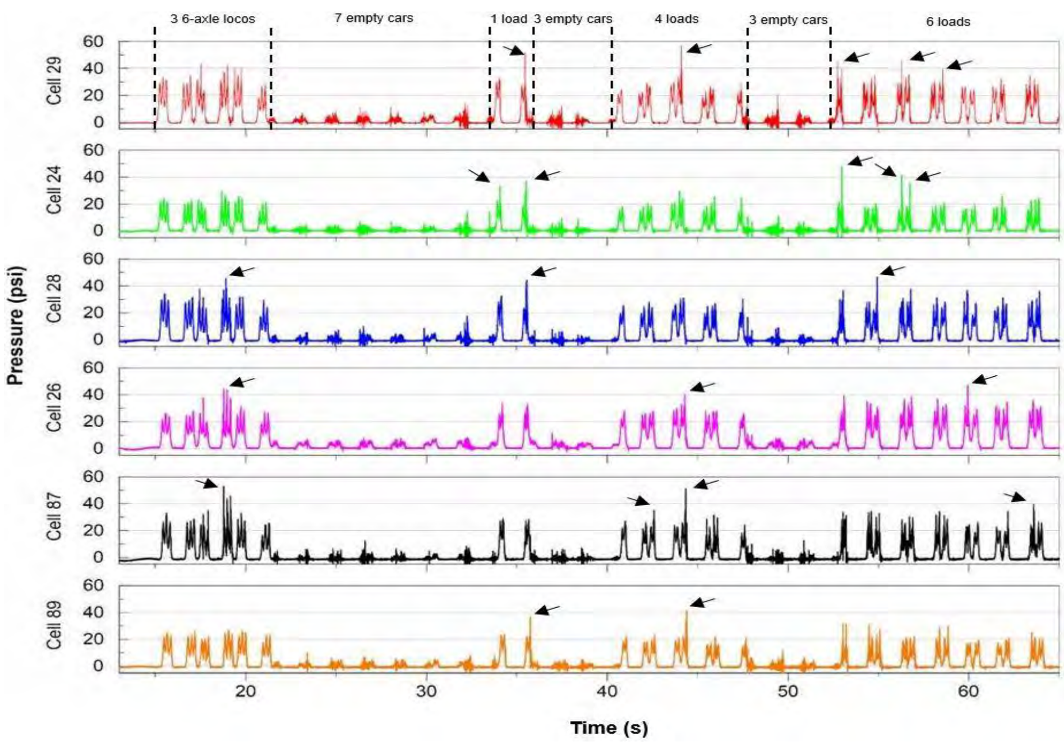


Figure 8a. Pressure traces for the forward portion of a typical mixed freight train passing over pressure cells positioned under six consecutive ties at the tie/ballast interface. The top legend notes the types of vehicles. The locomotives are on the left side; train is traveling from right to left.

The six cells were positioned under six consecutive ties under a single rail. As noted in the legend at the top of the figure, the pressures for the three 6-axle locomotives are shown on the left side of the plots for the six cells as six pressure peaks, each consisting of three sub-peaks. Following the locomotives are seven empty cars having substantially lower pressure peaks for the fourteen wheels. The empty cars are followed by a single loaded car having pressure peaks similar to the locomotives, as expected, since the individual wheel loads are similar in magnitude. Additional empty and loaded cars are follow for a total of twenty-four freight cars.

For a given wheel loading, individual cells record slightly different pressures, primarily due to the effect of variable compaction or density of the underlying ballast under individual ties, which also affects the support conditions. The greater the ballast is compacted under a given tie, the higher will be the pressure transferred by that tie to the ballast due to the increased support (2). These differences in pressure readings remain remarkably similar over a period of time. If the track is disturbed during a surfacing or tamping process, the relative distribution of pressures under individual ties will change slightly, but the average pressure will be similar to the average pressure before the track support was altered due to adjustments in the ballast compaction.

Note that the typical tie/ballast pressures under the locomotives range from 20 to 30 psi (140 to 210 kPa) and is consistent for a given locomotive. Locomotive wheel loads are typically 36,000 lbf (161.4 kN). Similar pressure values are measured for heavily loaded freight cars having typical wheel loads of 36,000 lbf (161.4 kN). Empty freight cars have typical wheel loads of 6,000 lbf (26.9 kN) producing pressure values about 6 psi (41 kN). These values are similar to those reported in previous documented findings (2).

The arrows point to wheels producing excessive pressure values, in excess of nominal pressures recorded for assumedly smooth wheels. These increased dynamic loadings can produce tie/ballast pressures twice or more of the values for the pressures exerted by smooth wheels.

Figure 8b contains enlarged views of the pressure versus time plots highlighting pressures recorded by Cell 87 for the three locomotives and groups of four empty cars, and four loaded cars. The pressures increase as the wheel approaches a given tie. The pressure peak is a maximum the instant the wheel is directly above the particular tie. A portion of the wheel load is carried by adjacent ties. Note the abnormally high peaks above the nominal pressures for smooth wheels. These are indicative of high-impact wheels.

Figure 8c contains similar plots for the initial portion of an intermodal train. The particular train consists of three 6-axle locomotives followed by loaded, primarily articulated, intermodal cars, most consisting of either 8 or 12 axles supporting the various connections and portions of the cars. The individual wheel loads vary depending on the spacing of the trucks and sharing of loadings by adjacent cars. As noted on the expanded plot below for a typical articulated car, typical tie/ballast pressures range from 8 psi (55 kPa) for lightly loaded wheels, to 20 psi (140 kPa) for intermediately loaded axles, to 35 psi (240 kPa) for heavily loaded axles.

SECTION 9: WILD WHEEL PARAMETERS AND TRACKBED PRESSURE RELATIONSHIPS

WILD wheel loading data obtained from either the Ebenezer or Flatrock detectors was compared to trackbed pressure data for the same trains traversing the Mascot trackbed pressure cell test site. Various measured WILD parameters were compared to recorded trackbed pressures. These were direct wheel-to-wheel comparisons. Cursory reviews of the digitized WILD and pressure data revealed likely relationships as the recorded WILD wheel loading magnitudes increased, the recorded trackbed pressures at the tie/ballast interface also increased.

The top trace in Figure 9a shows the relationship for the 46 loaded cars of a freight train containing predominately loaded freight cars. The train traversed the WILD at 32 mph (52 km/h) and the pressure test site at 20 mph (32 km/h). The WILD parameter of Peak load is considered the best measure for identifying the presence of high-impact wheels (see Figure 6 and associated discussion). The pressure values represent the average for the six cells

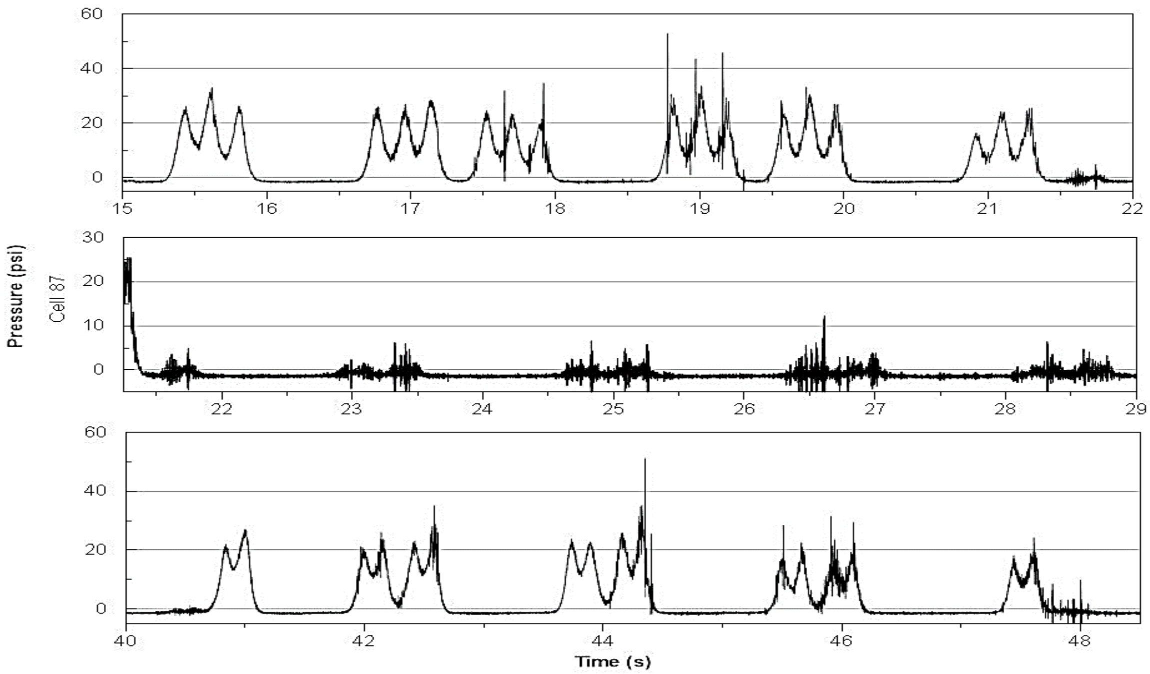


Figure 8b. Expanded view of pressure traces: top is the three 6-axle locomotives, middle is four empty freight cars, and bottom is four loaded freight cars. The abnormally high pressure peaks, above nominal pressures, are indicative of high impact wheels.

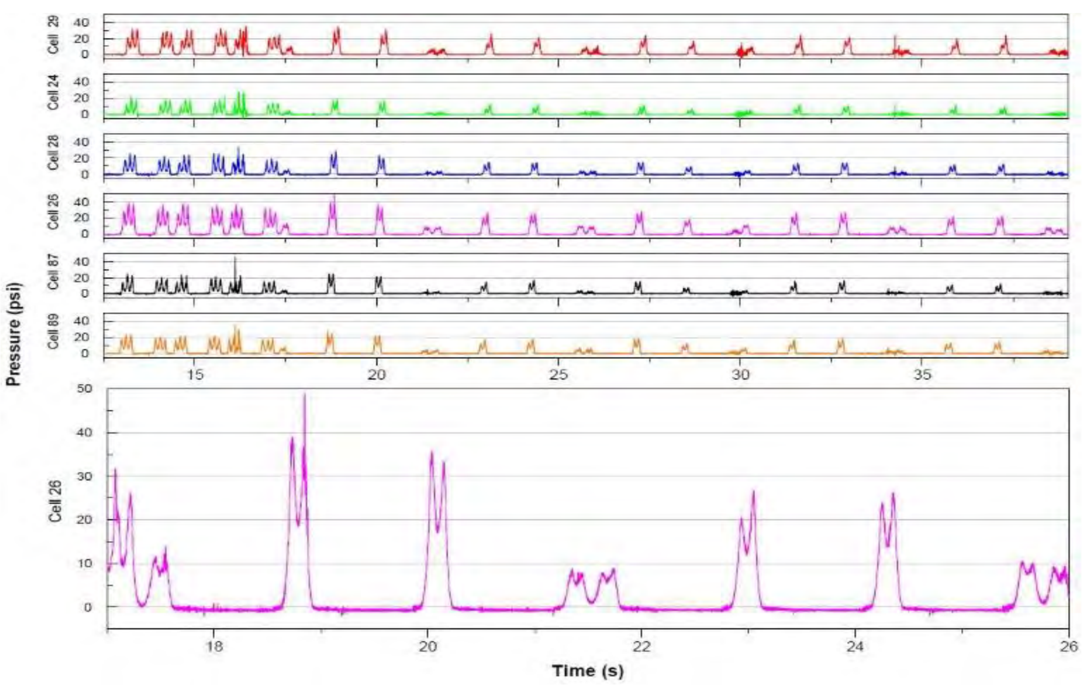


Figure 8c. Top view is forward portion of an intermodal train showing the pressures applied to the ballast as the train traverses the six cells. The three 6-axle locomotives are shown on the left with a portion of the intermodal cars following. The bottom view is an expanded pressure trace for several wheels of the intermodal cars.

As indicated, the increases in Peak values relate favorably to increases in recorded trackbed pressure. The relationship has a R-squared of 0.68. Based on the regression relationship, an increase of 36% in Peak loading results in an 81% increase in ballast pressure.

The bottom trace in Figure 9a shows the relationship between the WILD Nominal values and trackbed pressures. The R-squared was slightly higher with a value of 0.81. An increase of 29% in Nominal loading results in an 81% increase in ballast pressure.

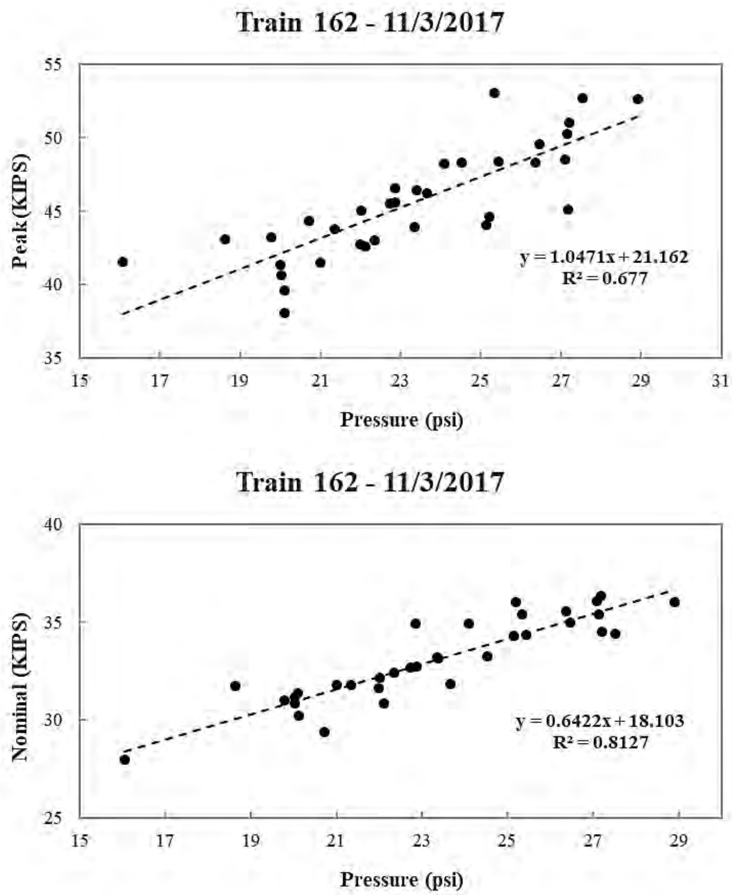
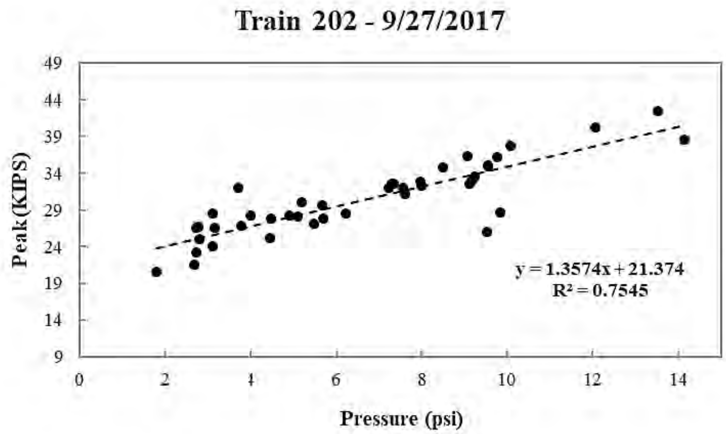


Figure 9a. Relationships between WILD Peak and Nominal wheel loadings and tie/ballast pressures for loaded freight cars.

Attempts to relate WILD parameters to trackbed pressures for empty cars were only marginally successful. The pressure differentials from wheel-to-wheel vary slightly in magnitude and the maximum pressure peaks are difficult to precisely identify. The pressure variability for a given wheel can be observed in Figure 8b, middle trace. Note the jagged variable shape of the empty car pressure peaks. This can be compared with the reasonably smooth shape of the pressure peaks for the locomotives (top) and loaded cars (bottom).

Figure 9b shows the WILD Peak force and Nominal force relationships to the measured ballast pressures for an intermodal train traveling at 35 to 40 mph (56 to 64 km/h), the same speed as when traversing the pressure test site. The wheel loads varied significantly, with average loadings less in magnitude than heavily loaded freight cars. Normally these types of trains are considered to have fewer high-impact wheels than the heavier-loaded mixed and unit freight trains. The R-squared was 0.75 for the Peak/pressure relationship and 0.83 for the Nominal/pressure relationship. This further substantiates the positive relationship between WILD loadings and companion tie/ballast interfacial pressures levels.



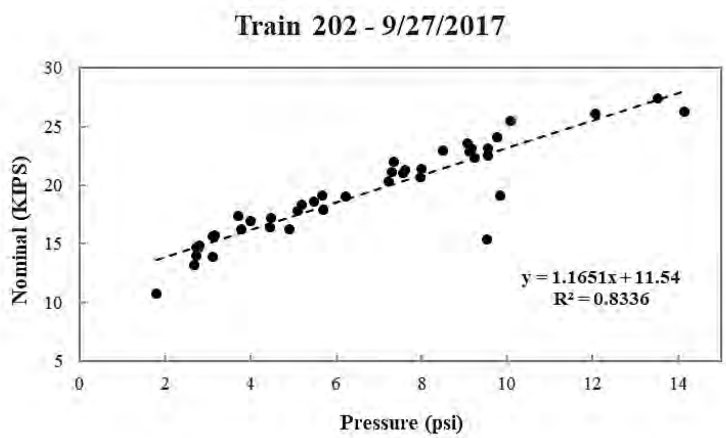


Figure 9b. Relationship between WILD Peak and Nominal wheel loadings and tie/ballast pressures for intermodal cars.

Further evaluation of the relationship between Nominal Force WILD measurements and corresponding tie/ballast interfacial pressures is shown in Figure 9c. Data for a combination of six trains – one empty unit coal, one intermodal, and four mixed freights consisting of empty and loaded cars -- is included. Data for the locomotives is also included. The data points represent the values for each wheel, rather than the averages per car as shown for the trains in Figures 11 and 12. The relationship indicates an R-squared of 0.87. The predominance of nominal wheel loads is in the 30 to 37 kip (133 to 164 kN) range, typical for loaded high capacity freights cars and locomotives.

For wheel loads below about 9 kips (40 kN), tie/ballast pressures, although low, vary significantly, as shown in Figure 9c. The primary reason the pressures vary as a given when passes over a cell is that the pressure trace is very jagged and it is difficult to determine an accurate average pressure at these low pressure levels. A typical trace for an empty car is shown in the middle trace of Figure 9. These pressures vary over a range from 1 to 9 psi (7 to 62 kPa); levels primarily produced by empty cars. The higher values in this range represent high impact empty car wheel loads.

Figure 9d contains the same data as Figure 9c except data for nominal loads less than 9 kips (40 kN) was excluded from the analysis. The remaining data represents loaded cars and locomotives with the bulk being nominal wheel loads between 30 to 40 kips (133 to 178 kN). The relationship indicates an R-squared of 0.92. The measured tie/ballast pressures for loaded cars and locomotives are better delineated than that for empty cars. This explains the slightly higher relationship between WILD Nominal loadings and corresponding tie/ballast pressures considering only the heavy wheel loads.

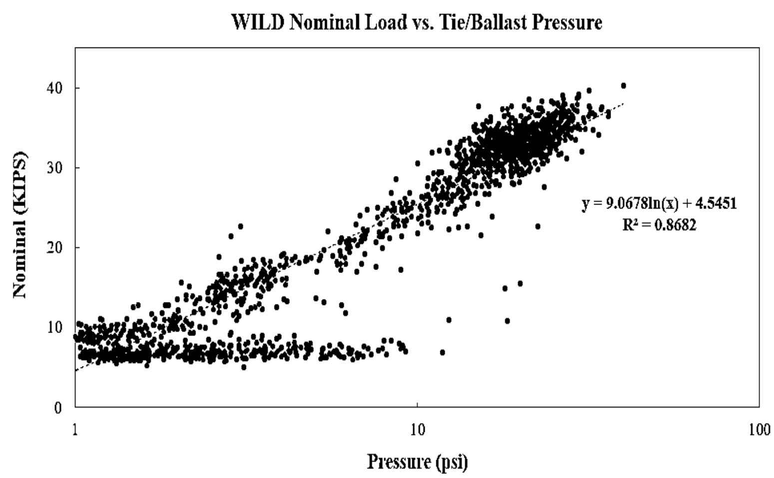


Figure 9c. Relationship between WILD Nominal wheel loadings and corresponding tie/ballast pressures.

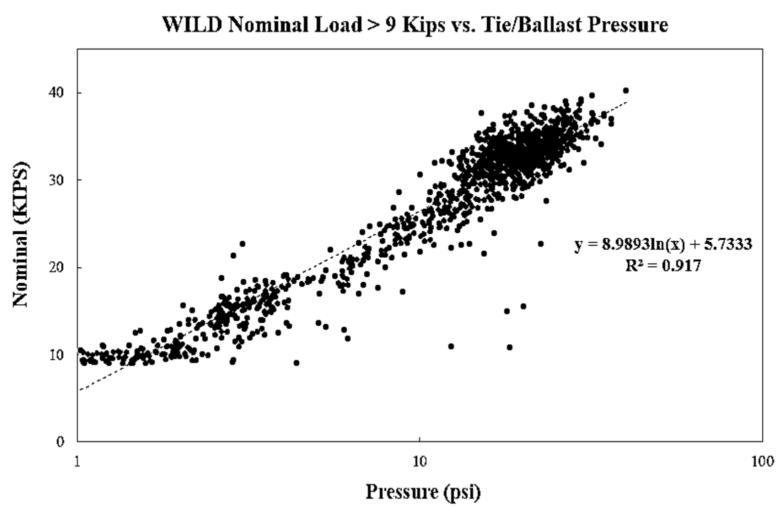


Figure 9d. Relationship between WILD Nominal wheel loadings (greater than 9 Kips) and corresponding tie/ballast pressures.

SECTION 10: RESULTS AND DISCUSSION

This report described the development of a method to measure average railroad track tie/ballast interfacial pressures using pressure cells specially designed for granular materials. The validity of the test method was verified with a series of laboratory tests using controlled loading magnitudes on prototype sections of trackbed with specially designed resilient frames/boxes for simulating typical in-track loading conditions. The selected procedure provided excellent correlation between controlled applied laboratory machine pressures and simultaneously measured cell pressures. The cells were recessed within the bottom of the ties for both the laboratory verification tests and subsequent in-track revenue train tests.

For the in-track tests, a series of cells were positioned under the rail at successive tie/ballast interfaces. Trackbed pressure measurements were conducted for numerous revenue freight trains during several months. After raising and surfacing the track, following installation of the instrumented ties, the track (mainly ballast) was permitted to further consolidate under normal accruing train traffic to insure that the ballast was tightly and uniformly compacted under the ties to effect transmission of equalized pressures from each of the ties to the ballast.

Measured pressures, directly below the rail/tie primary influence area at the tie/ballast interface, are considerably lower than trackbed pressures previously assumed for locomotives and loaded freight cars. These measured pressures range from 20 to 30 psi (140 to 210 kPa) for smooth wheels producing negligible impacts.

WILD wheel loading magnitudes obtained from nearby WILD wayside detectors were compared to trackbed pressure data for the same trains traversing the trackbed pressure cell test site. Various measured WILD parameters were compared to recorded trackbed pressures for loaded freight cars. The results indicate that increases in Peak wheel load values relate favorably to increases in recorded trackbed pressures. As an example, based on the regression relationship, as the Peak loadings increase -- the tie/ballast interfacial pressures increase by a factor of 2.25. Similar results were obtained comparing increased Nominal wheel loadings and increased tie/ballast interfacial pressures; pressures increased by a factor of 2.79.

The positive relationship between WILD loadings and corresponding tie/ballast interfacial pressure levels was further substantiated by comparing WILD Peak force and Nominal force relationships to measured tie/ballast pressures for an intermodal train. A R-squared value in the range of 0.8 was obtained for a variety of wheel loads.

The relationship between WILD Nominal wheel loading and tie/ballast pressure measurements is quite good for the higher magnitudes of wheel loadings. An R-squared value of 0.92 was obtained for a group of trains, excluding the light (empty car) wheel loads, and only considering the loaded car and locomotive wheel loads.

Higher wheel load magnitudes and increased dynamic impacts due to imperfections in wheel-tread surfaces increases the magnitude of the pressures correspondingly transmitted to the trackbed support as indicated by increased in-track pressures measured at the tie/ballast interface. The added increase in tie/ballast pressure, due to the impact of imperfect wheel surfaces, basically serves in a similar manner as increasing the nominal permitted wheel loads. The resulting increased dynamic impact forces can contribute to higher degradation rates for the track component materials and more rapid degradation rates of the track geometry.

SECTION 11: ACKNOWLEGEMENTS

This research was primarily funded by the National University Rail Center (NURail), a U.S. DOT OST Tier 1 University Transportation Center. Graduate Assistant Macy L. Purcell and Visiting Professor Qinglie Liu assisted significantly during the early phases of the research. Also appreciated are the financial assistance, cooperation, and significant contributions of Norfolk Southern Railway Company with the track site installations activities and continued monitoring and logistical aspects of evaluating the WILD data. Special appreciation is extended to LB Foster (Salient Systems) Company for their technical expertise, exportation of data, and review of the data processing.

SECTION 12: REFERENCES

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Modeling Crosstie-Ballast Load Distribution in a Railroad Trackbed via Finite Element Analysis

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National University Rail Center - NURail
US DOT OST-R Tier 1 University Transportation Center

TECHNICAL SUMMARY

Title

Modeling Crosstie-Ballast Load Distribution in a Railroad Trackbed via Finite Element Analysis

Introduction

This report outlines the procedure used to perform a Finite Element Analysis using RISA 3D on a railroad timber crosstie trackbed superstructure. The model predicts the load distribution at the crosstie-ballast interface, with results compared to previously collected in-track measurements. A brief summary of current research being conducted at the University of Kentucky provides the background to the modeling approach taken herein.

Derivation of RISA 3D program input variables are outlined in this report. In-track measured values are compared with RISA's predicted values. Following that is a discussion of early troubleshooting and the corresponding problems encountered. A sensitivity analysis performed on the main model assisted in determining the extent of the assumed linearity of the model's trackbed superstructure support. A separate RISA analysis analyzed the effects of a hypothetical poor/muddy trackbed support condition.

Approach and Methodology

Based on previously conducted experiments, the authors successfully measured lateral load distribution along a crosstie in a laboratory setting and collected in-track measurements of longitudinal load distribution between adjacent crossties. This data was obtained via the use of granular pressure cells recessed into timber crossties and later used to create a graphical representation of the measured load distribution of a track consisting of a half-length crosstie section. The load distribution obtained from this data was used as the basis for structural modeling.

The track superstructure model was comprised of conventionally sized, timber crossties each 7 in. x 9 in. x 8.5 ft. (180 mm x 230 mm x 2.6 m) in size and assigned red oak wood material properties. The steel rail was defined as a Generic Shape with assigned section and material properties obtained from the 2017 AREMA (American Railway Engineering and Maintenance-of-Way Association) manual. Crosstie spacing was set at 21 in. (0.533 m) center-to-center. A combination of one-way compression springs and two-way springs were used as the crosstie-ballast subgrade support.

RISA results were compared to the measured results and a sensitivity analysis was conducted to analyze the extent of the assumed linearity of the subgrade springs. In addition, a poor/muddy trackbed support scenario was considered by modifying the subgrade spring constants to reflect a track modulus representative of a poor/muddy trackbed support.

Findings

The measured graphical results compared very well with the model's predicted values as will be discussed in this report. The sensitivity analysis showed that the linear subgrade springs were accurate within the +10% to -10% range of the assigned spring constant value, but still respectable within the +20% to -20% range.

The poor/muddy trackbed model showed an increase in load distribution underneath the crossties with more adjacent crossties engaged in load carrying than when considering normal subgrade conditions (i.e., the trackbed load carrying distribution appears to engage additional ties in load carrying with ever decreasing quality of subgrade support).

Conclusions

This report outlines the procedure used to perform a Finite Element Analysis on a timber crosstie trackbed superstructure using RISA 3D to recreate and predict the load distribution at the crosstie-ballast interface. Based on prior in-track pressure measurement research performed at the University of Kentucky, close estimates obtained for the crosstie-ballast pressure and load distribution were used as the basis for the constructed Finite Element Model. It is unclear whether any significant degrees of structural modeling for analysis purposes are used for trackbed design and evaluation. Nor is it clear if the extent of the load carrying and distributing capabilities of a railroad track is fully understood. The modeling approaches contained herein could potentially be used for future designs or analyses.

Results from the RISA model reference just one set of data. It is reasonable to assume that the model's output may vary slightly if data from other sources are used. However, the scale of difference is small, with most of the variability arising at the pressure cells directly under the rail. The reference data used in this report provides a reasonable estimate for the extent of the load distribution.

In-track measured pressure cell values compared favorably with the predicted results given by the main RISA trackbed model. Deviation between respective values was minor, granting added confidence in the RISA model.

Poor/muddy track support conditions used the same modeling approach as that in the main model. It would be interesting to compare these results with actual poor/muddy track data. Results from this poor/muddy in-track support data could be compared to either the results from this report's modeling approach or another finite element approach.

The spring constants used in this report would be specific to the layout or location of the underlying pressure cells in the timber crossties as detailed herein. It would be interesting to determine how much variability these spring constants would experience if the locations of the pressure cells were changed.

Recommendations

The modeling method described herein is idealized for a fully tamped, ballast trackbed support and is inherently limited to the initially assumed Track Modulus (conventionally 3000 lb./in./in. (20,684 kN/m/m) for tamped, wood crosstie track) (assumed 500 lb./in./in. (3,447 kN/m/m) value for muddy track). The selected Track Modulus value is used to determine deflection values used to compute the approximate linear spring constants for use in the model. The consistent use of these target deflections to obtain values for the subgrade spring constants suggests that a railroad trackbed is not linear as modeled herein, but is likely nonlinear.

The approach detailed in this report provides an adequate working model to closely estimate the load distribution at the crosstie-ballast interface, but in future renditions of this research, perhaps using a program capable of utilizing nonlinear springs, such as SAP2000, could provide improved and more accurate results.

Publications

Thompson, B.D. “Modeling Crosstie-Ballast Load Distribution in a Railroad Trackbed using a Linear-Elastic Analysis”. Graduate Research Report, Department of Civil Engineering, University of Kentucky, May 2019.

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1) SECTION 1. ABSTRACT

This report details a linear-elastic approach to modeling crosstie-ballast interfacial load distribution in a railroad timber crosstie trackbed using RISA 3D, a finite element modeling software. RISA results closely mirrored experimental findings with some slight deviation as discussed in the report. The modeling methodology outlined herein advances previous trackbed research at the University of Kentucky and could be used in future trackbed finite element analyses for design or research in load path distribution and load magnitude predictability.

This report details the approach used to model a timber crosstie trackbed superstructure, including initial troubleshooting stages, and explains the derivation of program input variables. A sensitivity analysis performed on the main model determined the workable extent of linearity. The analysis also examined the effect of a poor/muddy trackbed support loading condition. The report concludes with potential recommendations regarding the work.

Keywords: Crosstie-Ballast Interfacial Load Distribution, RISA 3D Modeling, Timber Crosstie Trackbed Superstructure, Sensitivity Analysis, Railroad Trackbed

2) SECTION 2. INTRODUCTION

At the time of writing, current track design procedures reference the AREMA (American Railway Engineering and Maintenance-of-Way Association) manual for railway design guidelines. A.N. Talbot and his committee were responsible for deriving the empirical equations and load distribution methodologies portrayed in this manual. These equations, generally referred to as the Talbot Method, make various assumptions and require inputs such as wheel diameter and dynamic impact factors, which may not be as relevant in modern day applications. Based on these assumptions and data from current research, the accuracy of the Talbot Method is unclear in today's modern, heavy freight loadings and larger size continuously welded rail. It is also unclear whether any North American freight railroads or consultants use any form of finite element modeling in design; opting to simply rely on various AREMA recommendations and with what has generally worked throughout the years (i.e., For Class 4 track: 12 in. (305 mm) of ballast, 136 RE rail, 21 in. (533 mm) crosstie center-to-center spacing, etc.).

Current work being conducted at the University of Kentucky has preliminarily disproven some of Talbot's assumptions such as that only the outer third of a crosstie is in bearing with the ballast (1). It is common procedure for the Railroads to only tamp the ballast underneath the outer thirds of the crosstie to prevent cambering in the crosstie's middle section due to settlement of the outer portions of the crosstie when under repeated loading (2). It will be shown in this report that the middle portion of the crosstie does, in fact, carry a small, but notable, fraction of the load, roughly 2.5% to 3% despite being untamped.

Like most structural designs, it would be advantageous for an engineer to be able to predict within a reasonable tolerance, how such said structure will behave under specified loading and trackbed support scenarios. The modeling approach, as detailed herein, seeks to do just that. Load distribution is a vital component to accurately analyze any structural component of the system as a whole or in part. It is possible that either this or a similar modeling approach could be utilized in the design of railway trackbeds rather than simply relying on a site-specific trial and error approach.

This report outlines the procedure used to perform a finite element analysis using RISA 3D on a timber crosstie trackbed superstructure. The model predicts the load distribution at the crosstie-ballast interface, with results compared to previously collected in-track measurements. A brief summary of current research being conducted at the University of Kentucky provides the background to the modeling approach taken herein.

Derivation of RISA 3D program input variables is outlined. Following that is a discussion of early troubleshooting and the corresponding problems encountered. A sensitivity analysis performed on the main model helped determine the extent of the assumed linearity of the model's trackbed superstructure support. A separate RISA run analyzed the effects of poor/muddy trackbed support.

The report concludes with final comments and recommendations regarding the approach taken and a few concerns regarding input variables that may be of importance for future finite element modeling research.

3) SECTION 3. BACKGROUND

In earlier work, the authors conducted laboratory experiments to measure lateral load distribution along a crosstie and collected in-track measurements of longitudinal load distribution between adjacent crossties (1, 3, 4). This data was used to create an estimated load distribution of a track consisting of a half-length crosstie section. The methodology is presented below.

3.1 Lateral Load Distribution along a Half-Length Crosstie

The average lateral load distribution along a half-length crosstie was measured in the laboratory. A Baldwin/Satec hydraulic testing machine having a test range limit of 300,000 lbf (1,345 kN) applied loads. A conventional, timber crosstie 7 in. (180 mm) thick, 9 in. (230 mm) wide, and 55 in. (1.4 m) in length was used with four, 8 in. (203 mm) diameter by 1 in. (25.4 mm) thick Geokon Model 3515 granular materials pressure cells each capable of measuring an applied pressure up to 360 psi (2.48 kPa). These pressure cells were recessed flush with the bottom of the timber crosstie via machine routing. Care was taken to fasten the pressure cells in the crosstie recesses via screws or corner braces. Four cells labeled 68 through 71 were encased inside the timber crosstie as shown in Figure 3.1. A 55 in. (1.4 m) long crosstie was used instead of a 51 in. (1.30 m) (half the length of a 102 in. (2.60 m) conventional crosstie) so that the 8 in. (200 mm) diameter pressure cell would be fully enclosed and that the center of the pressure cell would be positioned at the full-length crosstie's centerline (3, 4).

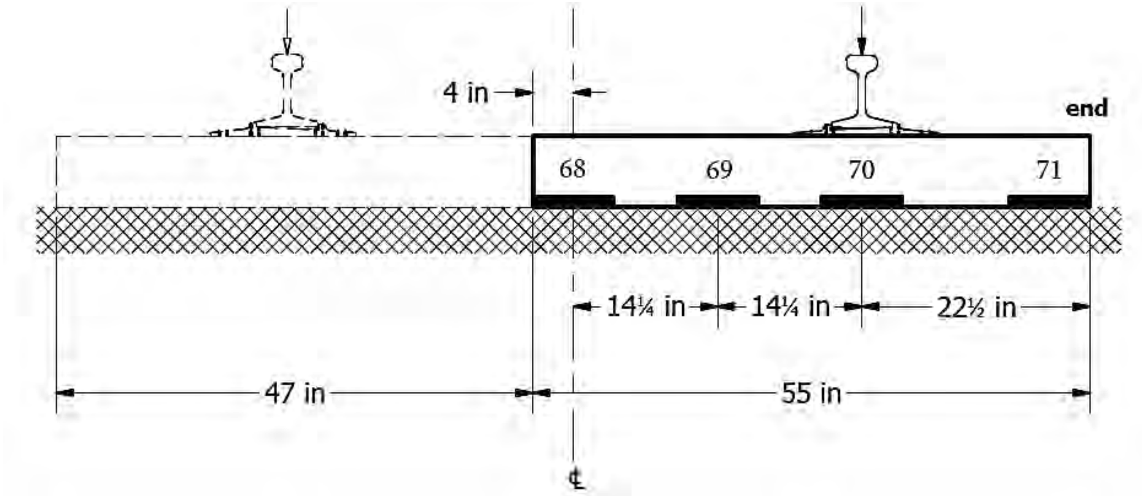


FIGURE 3.1. Cross-Sectional View of Half-Length Crosstie Pressure Cell Placement [1.0 in. = 25.4 mm] (4)

The timber, half-length crosstie was placed on a carefully constructed, granular trackbed comprising of a 3/16 in. (4.75 mm) thick layer of rubber mat on the bottom to simulate resiliency and artificially reach a typical total track deflection value of 1/4 in. to 1/3 in. (6.35 mm to 8.5 mm). A deflection target was used because Track Modulus is taken as 1 kip per inch of deflection per inch of rail (5). The mat was covered with a 4 in. (100 mm) thick layer of compacted, dense-graded, limestone subballast and a 14 in. (355 mm) thick ballast layer comprising of new and worn AREMA #4A graded granite of 2 in. (50 mm) or less in size for ease of ballast uniformity and crosstie seating. The rail section used was a 10 in. (255 mm) length of conventional 136 RE rail positioned on a standard 14 in. (355 mm) long by 8 in. (203 mm) wide steel tie-plate. To ensure a direct vertical load, a 1/8 in. (3 mm) thick shim placed along one side of the rail seat in the tie plate negated the seat cant angle. An image of the setup is shown in Figure 3.2.

FIGURE 3.2. Half-Length Crosstie in Laboratory Setup (4)

The 8 in. (203 mm) diameter Geokon Model 3515 granular materials pressure cells contain hydraulic fluid encased between two steel plates welded together around the edges. As these pressure cells are loaded, the internal fluid pressure increases, prompting an attached pressure transducer to relay the measured pressure to a National Instruments Model NI 9203 C Series Current Input Module data logger attached to a laptop computer utilizing the LabVIEW software. These pressure cells are capable of measuring applied pressures ranging from 0 to 360 psi (0 to 2.48 MPa). However, experience with these instruments suggest that these cells are generally more accurate under higher applied loads, likely 1.0 to 1.5 kips (4.45 kN to 6.67 kN) and above.

Testing employed three sets of measurements comprising of static compressive loads applied to the rail in 1,500 lbf (6.7 kN) increments up to 10,500 lbf (47 kN). The pressure cells measured the applied load and from this data, the load distribution along the half-length crosstie was graphically plotted. Figure 3.3 shows the percentage of load obtained in each of the pressure cells. The 10,500 lbf (47 kN) curve was used as the reference value since it was the highest recorded loading. An average of the 3000 lbf to 9000 lbf (15.6 kN to 40.0 kN) loading curves would likely have been more representative of typical load distributions in a trackbed as loads within this range would correspond to a combination of low to high trackbed stiffness and wheels with few to many wheel irregularities such as flat wheels (i.e., high trackbed stiffness and/or flat wheels attract higher stresses and loads). Based on the authors' laboratory trackbed configuration, the applied loads on the half-length crosstie would give pressure cell readings similar to those measured in-track (1). The percent load distribution corresponding to the highest load is:

Cell 68: 3% -- Cell 69: 22% -- Cell 70: 51% -- Cell 71: 24%

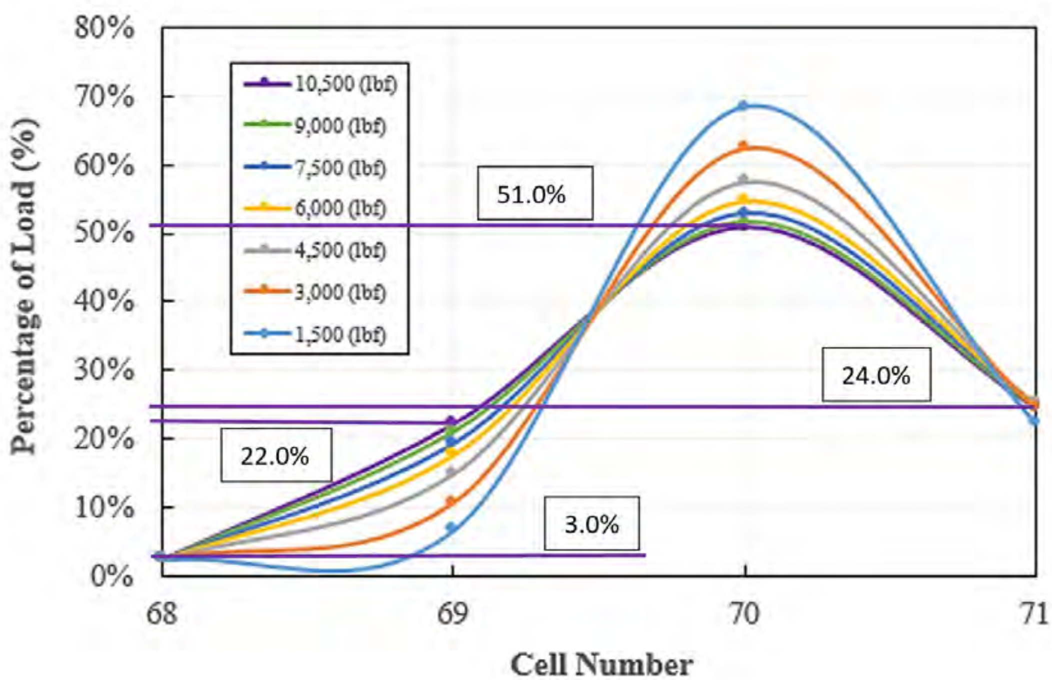


FIGURE 3.3. Percent Load Distribution in a Half-Length Crosstie [1,000 lbf = 4.45 kN] (4)

3.2 Longitudinal Load Distribution Between Adjacent Crossties

The average longitudinal load distribution between adjacent crossties was measured on a Norfolk Southern Railway revenue mainline in Mascot, TN. The line carries approximately 37 million gross tons (MGT) of traffic annually with a maximum train operating speed of 45 mph. The track consists of conventional 136 RE continuously welded rail, timber crossties, and granite ballast. An image of the track is shown in Figure 3.4.

FIGURE 3.4. Track View at Mascot, TN (1)

The pressure cell arrangement was similar to that in the laboratory, except that all pressure cells were recessed in the tie at the crosstie-ballast interface beneath the rail with the exception of two placed in the center of the crosstie (1, 3). Each pressure cell was connected to a datalogger and computer; the same as for the laboratory setting. The setup for the in-track pressure cells are shown in Figures 3.5a and 3.5b.

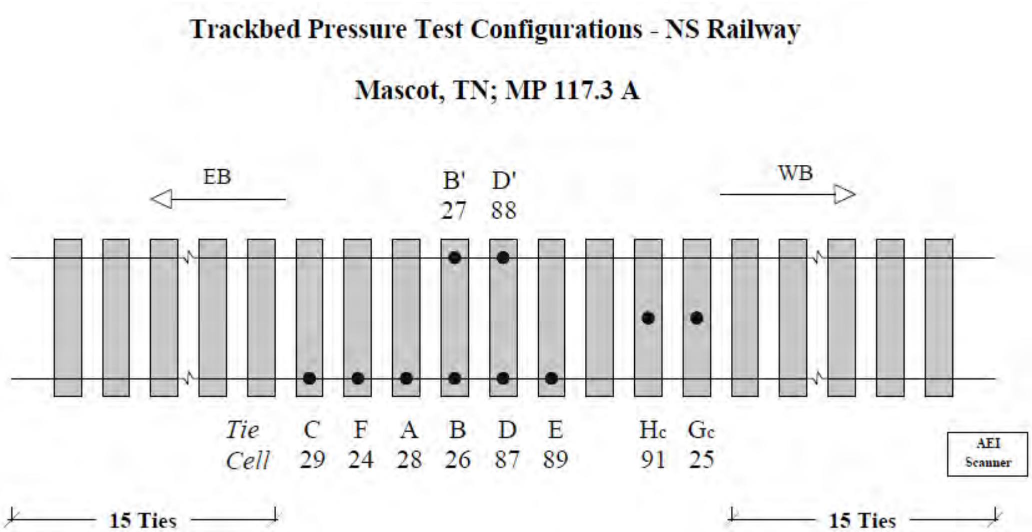


FIGURE 3.5a. In-Track Pressure Cell Setup (1)

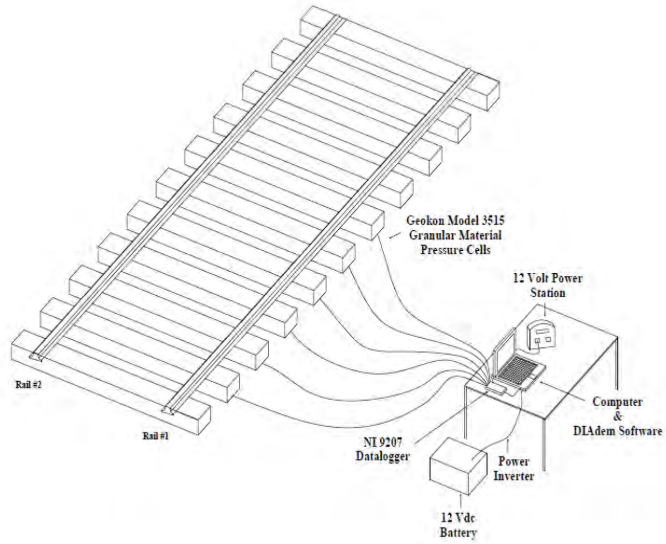


FIGURE 3.5b. In-Track Pressure Cell Setup (Computer Setup) (1)

Data referenced is from tests using the Federal Railroad Administration's Comprehensive Inspection and Test Train Consist which contained a conventional 6-axle Norfolk Southern locomotive, DOTX 218 test car, and DOTX 220 inspection test car. The DOTX 218 test car features a deployable axle capable of delivering variable loads from 2 to 22 kips (9 to 98 kN). Figures 3.6a, 3.6b, and 3.6c, depict the NS FRA consist.

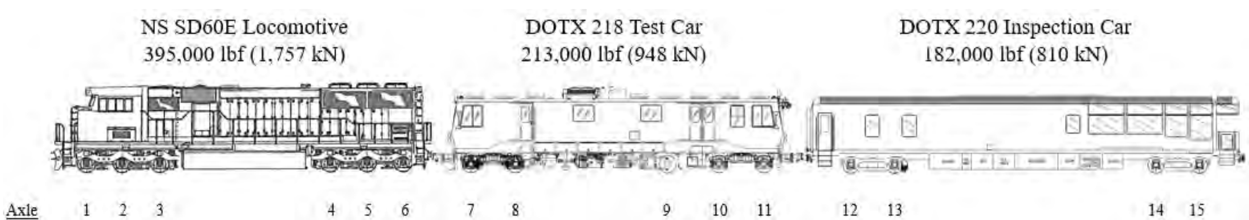


FIGURE 3.6a. NS FRA consist (1)

FIGURE 3.6b. NS FRA consist (DOTX 218 Test Car) (1)

FIGURE 3.6c. NS FRA consist (DOTX 218 Test Car – Deployable Axle) (1)

Four static loads were applied to the rail at 10, 15, 20, and 22 kips (44, 67, 89, and 98 kN). Figure 3.7 shows the corresponding loads on the underlying pressure cells as recorded and graphed. Note that the applied load is distributed over five crossties. The percent load distribution corresponding to the highest load is:

Crosstie D: 6% -- Crosstie E: 22.5% -- Crosstie F: 43% -- Crosstie G: 22.5% -- Crosstie H: 6%

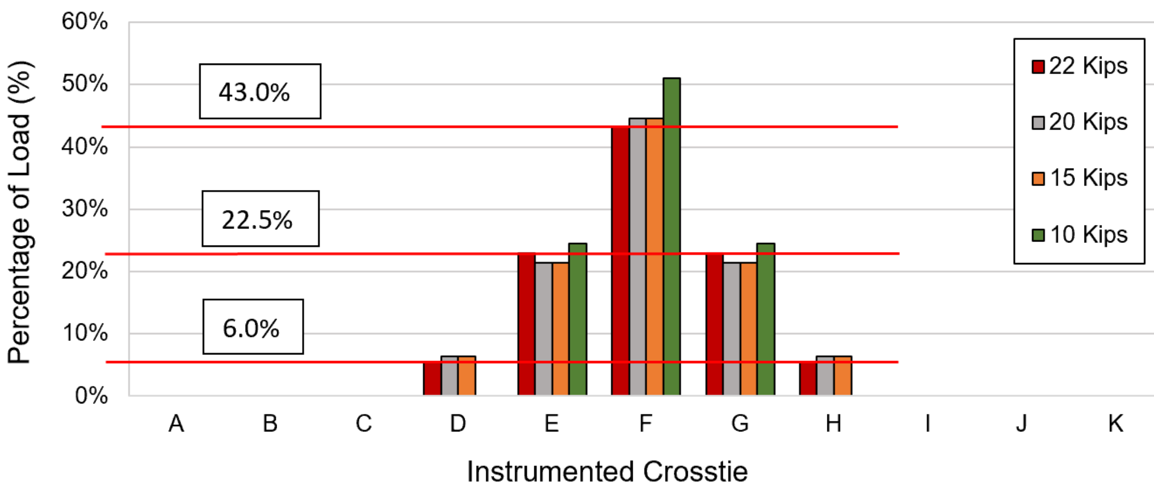


FIGURE 3.7. Percent Load Distribution Along Multiple Crossties (Wheel Above Crosstie F) [10 kips = 44.5 kN] (1)

3.3 <u>Half-Length Crosstie Load Distribution</u>

The lateral and longitudinal percentages of load distribution reflect the distributed loads from a simulated wheel load. The percentage of load applied at each pressure cell along the crosstie is the product of the corresponding load distribution percentages from the lateral crosstie and in-track longitudinal measurements (e.g., pressure cell underneath wheel = 43% x 51% = 21.93% of the total applied load). Figure 3.8a shows the results in terms of the percentage of load distribution at each pressure cell. The product of the load percentage at each cell and the applied 36 kip (160 kN) wheel load provides the applied load in kips at that cell (Figure 3.8b). These figures are used as the basis for the finite element modeling approach, using RISA 3D, as detailed in the subsequent sections.

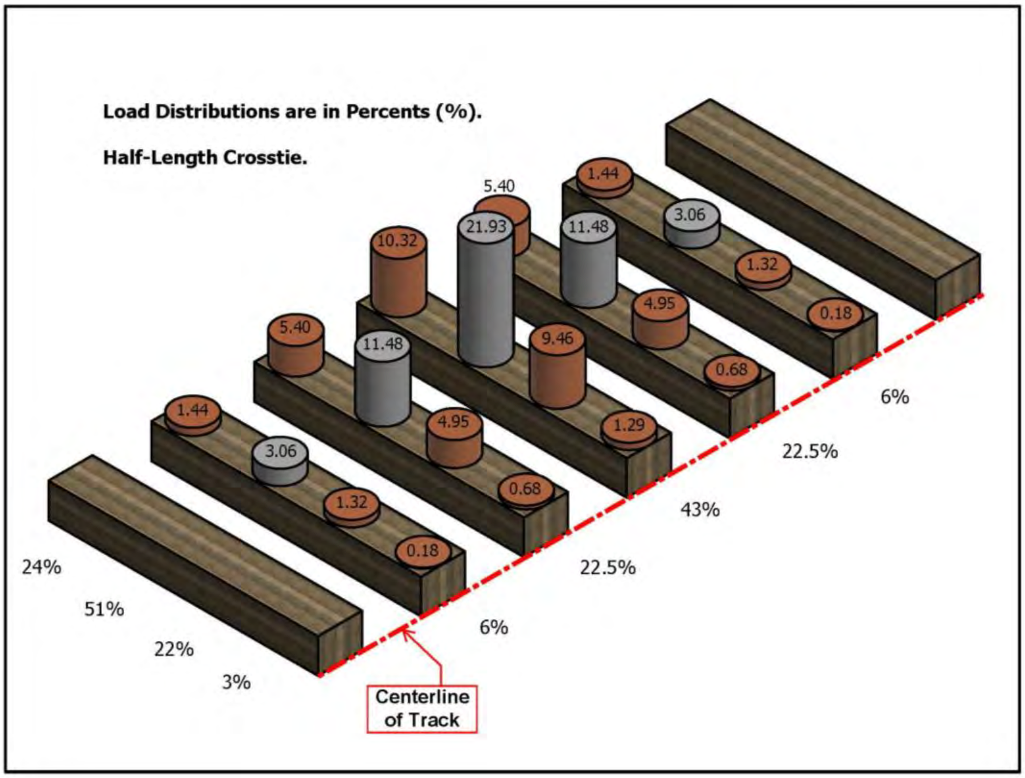


FIGURE 3.8a. Half-Length Crosstie Trackbed Percent (%) Load Distribution

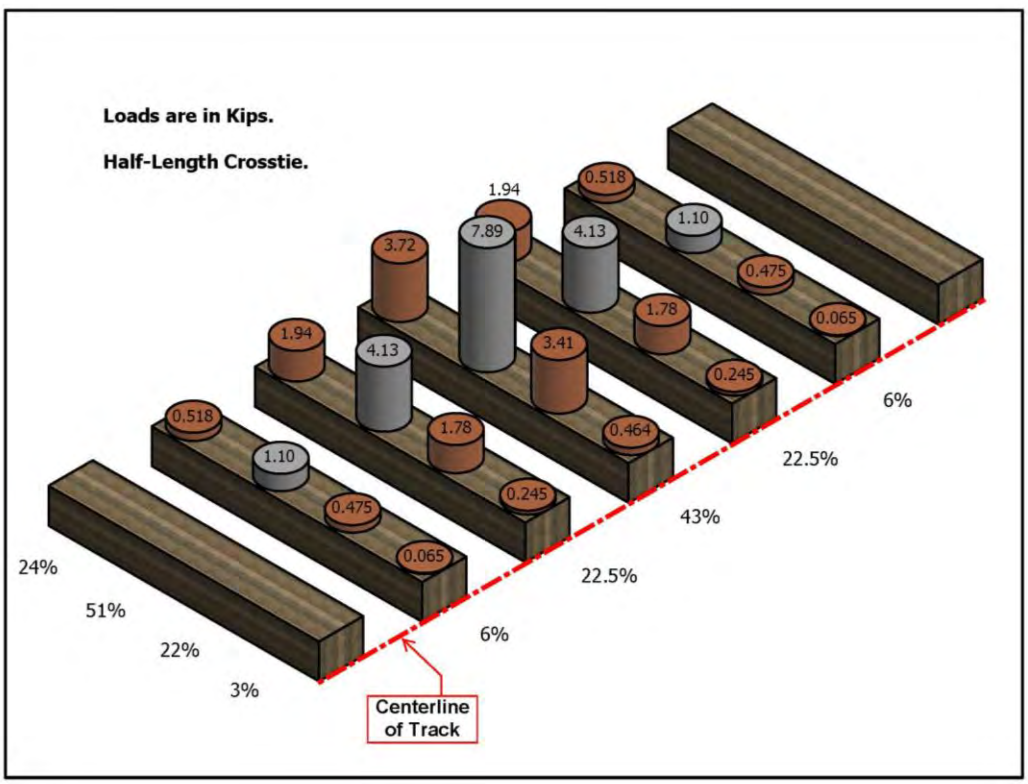


FIGURE 3.8b. Half-Length Crosstie Trackbed Load Distribution [10 kips = 44.5 kN]

4) SECTION 4. RISA 3D MODELING

Using RISA 3D (6), the authors developed a model of the track structure to permit analysis of crosstie-ballast interfacial loads (7). RISA programs are nodal based and make use of stiffness matrices to solve for loads and stresses. The approach requires user selected or defined nodes at locations deemed critical or of interest. RISA 3D does not natively include any railway materials or properties. In addition, because RISA 3D is a finite element modeling package, ballast and subgrade support cannot be directly modeled or entered into the program. To account for the lack of these internal preprogrammed functionalities, the authors created, defined, and applied Generic Elements with user defined material properties and a combination of two-way springs and one-way compression springs at defined model locations. Details of the model follow in the report.

4.1 The Approach

Many RISA runs (trials) were performed. This report details one of the most favorable results, how it was obtained, and how one might attempt to recreate the results. However, some steps may not be applicable to other finite element modeling programs.

4.2 Modeling of a Timber Crosstie

Typically, in the railroad industry, a standard timber crosstie trackbed has a Track Support Modulus of approximately 3000 lb./in./in. (20,684 kN/m/m) (5). Such track would experience about a 1/4 in. (6.35 mm) total track deflection under normal (36 kip) (160 kN) wheel loads, providing an ideal combination of strength, resiliency, and ride comfort (8). Straying too far from this deflection value may result in undesirable attributes such as an increase in dynamic loading effects. The authors used the 1/4 in. (6.35 mm) deflection value to determine spring constants used to simulate trackbed support in the model.

Timber crossties in the model are identical, representing a standard 7 in. x 9 in. x 8.5 ft. (180 mm x 230 mm x 2.6 m) size. The Young's Modulus of Elasticity of 1600 ksi (11,032 kPa) reflects a high-end average for Red Oak, a common timber crosstie species. The American Wood Council's National Design Specifications for wood construction lists varying values as low as 1000 ksi (6,895 kPa) for Red Oak (No. 2 Visually Graded, 5 in. x 5 in. (127 mm x 127 mm) and larger) (9), while some online engineering dedicated sources claim as much as 1800 ksi (12,411 kPa). A value of 1600 ksi (11,032 kPa) was deemed reasonable for use in this model.

Springs applied underneath the crosstie at locations representing the center of each pressure cell represent boundary conditions. The selected spring constants must reflect their tributary area. Two additional nodes are placed at each end of the crosstie to account for the full length of the crosstie since the center of the end pressure cells are 4 in. (100 mm) from the end of the crosstie.

The arrangement for a timber crosstie in RISA is shown in Figure 4.1, below. Nodes are placed at the ends and the center of each pressure cell. Vertical compression spring boundary conditions are placed at the spring-pressure cell interface where the springs are labeled as K1-K4. Horizontal two-way springs simulating crosstie-ballast resistance between adjacent crossties were placed at each nodal point. Tributary areas labeled A1-A4 (Figure 4.1) are calculated from the dimensions shown in Figure 3.1.

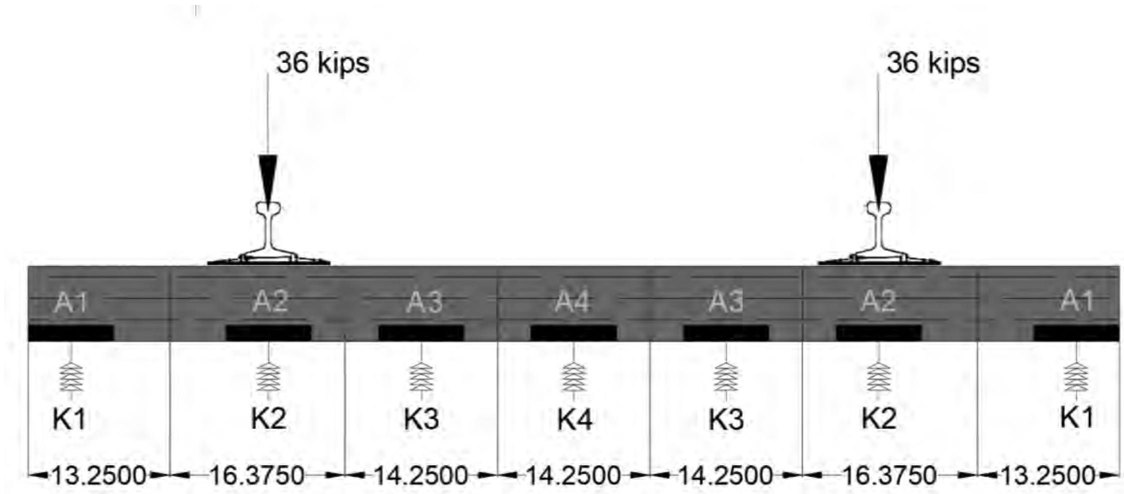


FIGURE 4.1. Crosstie Arrangement in RISA (dimensions in inches)
[10 in. = 254 mm] [36 kips = 160 kN]

4.3 Determining Pressure Cell Spring Constants

To determine the tributary area of each pressure cell, multiply the lengths shown in Figure 4.1 by a 9 in. (230 mm) crosstie width. This is the total area that each vertical compression spring must represent. The calculations, shown below, are rounded to three significant figures. By symmetry, these tributary areas are mirrored on each side of the center of the crosstie.

A1 = 13.25 in. x 9 in. = 119 in.^{2} (0.0768 m^2)
A2 = 16.375 in. x 9 in. = 147 in.^{2} (0.0948 m^2)
A3 = 14.25 in. x 9 in. = 128 in.^{2} (0.0826 m^2)
A4 = 14.25 in. x 9 in. = 128 in.^{2} (0.0826 m^2)

Using Figure 3.8a or Figure 3.8b, the initial spring constant estimates can be determined. Herein, Figure 3.8a will be used as the reference figure. Multiply the load distribution for each cell from Figure 3.8a by the applied load of 36 kips (160 kN), then divide by the ratio of the respective tributary area to the whole area. The upscaling ratio factor can be used with the area of half the crosstie rather than doubling the tributary area then dividing by the area of a whole crosstie (e.g., A1/0.5*A_{tie} instead of 2*A1/A_{tie}).

$A_{half\ tie}$ = Area of a Half-Length Crosstie = (102 in.) x (9 in.) x 0.5 = 459 in.^{2} (0.296 m^2)

K1 = 10.32% x (1/100) x 36 kips x 4 / (119 in.^{2} / 459 in.^{2}) = 57.3 kips/in. (10.0 kN/mm)
K2 = 21.93% x (1/100) x 36 kips x 4 / (147 in.^{2} / 459 in.^{2}) = 98.6 kips/in. (17.3 kN/mm)
K3 = 9.46% x (1/100) x 36 kips x 4 / (128 in.^{2} / 459 in.^{2}) = 48.8 kips/in. (8.55 kN/mm)
K4 = 1.29% x (1/100) x 2 x 36 kips x 4 / (128 in.^{2} / 459 in.^{2}) = 13.3 kips/in. (2.33 kN/mm)

Notes:

- Multiplying by 4 is the same as dividing by 1/4 in. (6.35 mm) total track deflection.
- Multiply 1.29% by 2 because Figure 3.8a represents the load from a half-length crosstie. The load at the middle pressure cell (A4, K4) would double because the other half of the crosstie must also be considered.

After many trial runs, it was clear that most of the load was centered around the middle three pressure cells, A3 and A4, as seen in Figure 4.1. Therefore, the spring constant values were multiplied by additional factors to propagate the load outward towards cells A1. It was eventually determined by trial and error that the following should be done:

- Multiply K1 by a factor of 7
- Multiply K2, K3, and K4 by a factor of 2

This gives the following final spring constants rounded to the nearest whole number:

K1 = 401 kips/in. (70.2 kN/mm)   K2 = 197 kips/in. (34.5 kN/mm)
K3 = 98 kips/in. (17.2 kN/mm)   K4 = 27 kips/in. (4.72 kN/mm)

The additional spring constant multipliers are the exact values required to grant the desired load distributions in the model. The initial estimates for the spring constants proved too small after confirming with several trial iterations as the load consistently gravitated inward toward the center of the crosstie rather than outward toward the end pressure cells. K1 was upscaled to a large degree (7 times) before preferable loads were attainable. This is likely because K1 is an additional 4.25 in. (108 mm) center-to-center from K2 than K3 is from K2 and because the ends of the crosstie are not fixed or pinned as idealized in conventional structural member ends and, therefore, do not have the necessary stiffness to attract the desired load.

K2 from Figure 4.1 falls just below the rail and comprised of the largest pressure cell tributary area. It was increased by a multiple of 2 to better match the data from Figure 3.8b. K3 and K4 from Figure 4.1 are interesting. Half of the pressure cell corresponding to spring constant, K3, is inside the zero-bearing zone as described by A.N. Talbot in the AREMA manual (middle third of the crosstie is not in bearing, untamped) (2). K4 falls entirely inside the zero-bearing zone and is theoretically, according to AREMA's analysis methodology, equal to zero. This research shows that the middle third of the crosstie is in bearing to some degree, though does not experience a large portion of the load compared with the outer thirds of a tamped crosstie. This can be seen from Figures 3.8a and 3.8b. Both cells were increased by a factor of 2 to better match the data from Figure 4b.

These additional spring constant multipliers indicate a disparity in deflection values between the laboratory setup, the in-track configuration, and the interpretation of values pulled from Figures 3.3 and 3.7. In order to negate any use of additional multiplication factors you would have to maintain the same total track deflection values across each platform whether in-track or in the laboratory under recreatable conditions. Due to the large degree of variability and the inherent nature of a non-homogeneous material such as ballast, this is a difficult undertaking.
The spring constant for the two-way springs representing the ballast confinement pressure in-between the crossties was determined using an approximate vertical ballast deflection of 0.15 in. (3.8 mm) (8).

The spring constant used was the 36 kip (160 kN) wheel load divided by this deflection. This would technically be vertical spring stiffness if only the deflection of the ballast below the crossties was considered. Regardless, it was used as the ballast-crosstie confinement spring constant because the actual horizontal deflection between crossties is unclear. The crosstie-ballast spring constant would, therefore, be:

K = 36 kips/0.15 in. = 240 kips/in. (42.0 kN/mm)

4.4 Modeling of the Rail

RISA does not have rail steel listed in its database and, therefore, the user must define the rail as a General Shape in RISA's Draw Members functionality. This feature lets the user enter any shape desirable provided the governing member's section and material properties are known. Figure 4.2 depicts the data entry box with the appropriate inputs referencing the section properties of conventional 136 RE rail obtained from the AREMA manual except for the Torsional Constant, J, and the Shear/Stress Factors (2, 10). The method for approximating the Shear/Stress Factors will be further discussed. Note that the Torsional Constant is somewhat irrelevant for this model, considering that the rail has such a short span and RISA does not consider the eccentricity between the steel wheel and the rail.

The Shear Area Factor for both axes is taken as the nominal web thickness times the overall depth of the rail divided by the total cross-sectional area of the rail. This ratio will determine the amount of the rail area that will take the applied shear load.

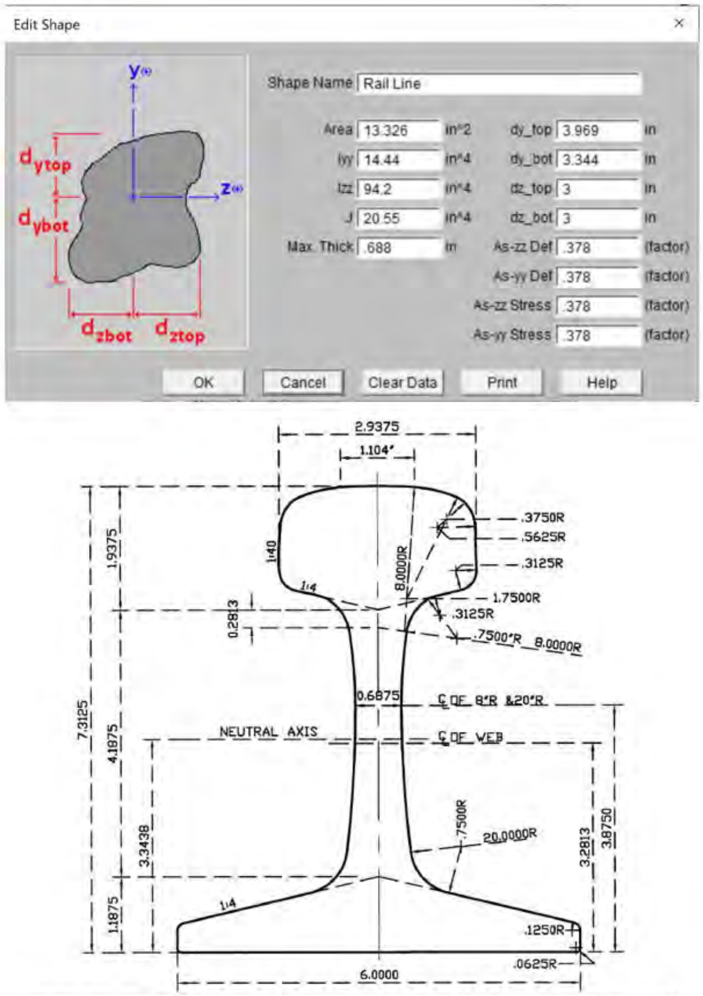
$A_{s-zz} = A_{s-yy} = (0.688 \text{ in. x } 7.31 \text{ in.}) / 13.3 \text{ in.}^2 = 0.378$

4.5 RISA 3D Viewport

Following the aforementioned procedure, a crosstie model was created. Only the end nodes were laterally restrained to represent ballast resistance around the ends of the crosstie. These nodes do not have a vertical compression spring as an additional boundary condition to account for the center of the pressure cell that is offset 4 in. (100 mm) inward from the edge of the crosstie. Apart from the end nodes, all other nodes are located at a location corresponding to the center of the 8 in. (203 mm) diameter pressure cells depicted in Figure 4.1.

The crossties are spaced at 21 in. (0.53 m) center-to-center per conventional mainline timber crosstie track. The final model was 49 ft. (14.9 m) in length, representing 29 adjacent crossties. Figures 4.3a and 4.3b respectively represent the unrendered nodal view and the 80% rendered view of the full model.

The rail is not offset from the crosstie nodes; rather it directly intersects the appropriate crosstie nodes (the center of gravity of the rail and crossties intersect). This idealized approach is discussed further in the Troubleshooting section.



1. Rail Area (square inch) Head Web Base Whole Rail	 4.8187 3.6375 4.8702 13.3263
2. Rail Weight (lb/yd) (based on specific gravity of rail steel = 7.84)	135.8826
3. Moment of Inertia about the neutral axis	94.20
4. Section modulus of the head Section modulus of the base	23.70 28.20
5. Height of neutral axis above base	3.34
6. Lateral moment of inertia	14.44
7. Lateral section modulus of the head Lateral section modulus of the base	9.83 4.82
8. Height of shear center above base	1.64
9. Torsional rigidity is 'KG' where G is the modulus of rigidity and K = (error for K greater than 10%)	6.24

FIGURE 4.2. Input Properties and Specifications of 136 RE Rail (2, 10)

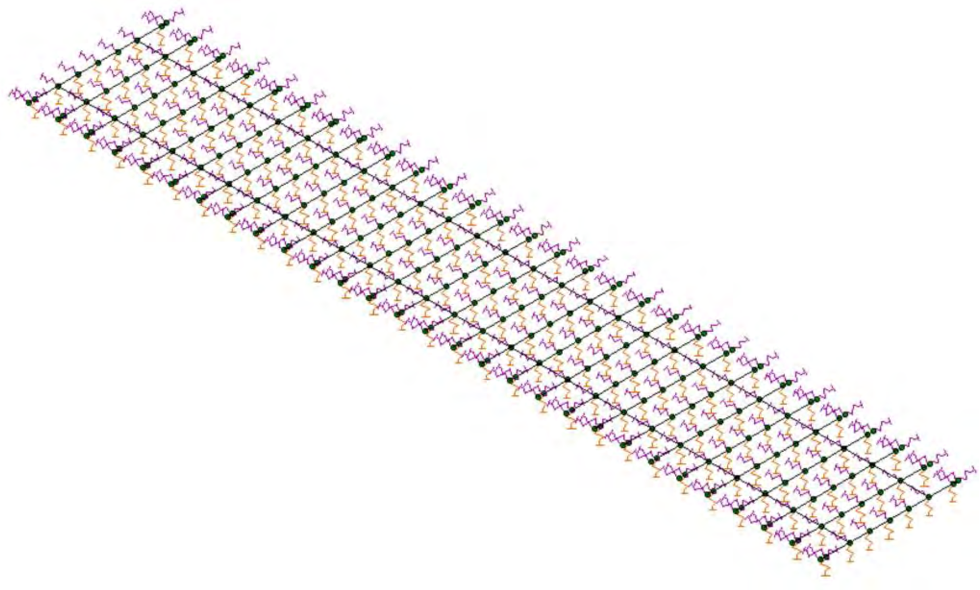


FIGURE 4.3a. Trackbed Model (Nodal View)

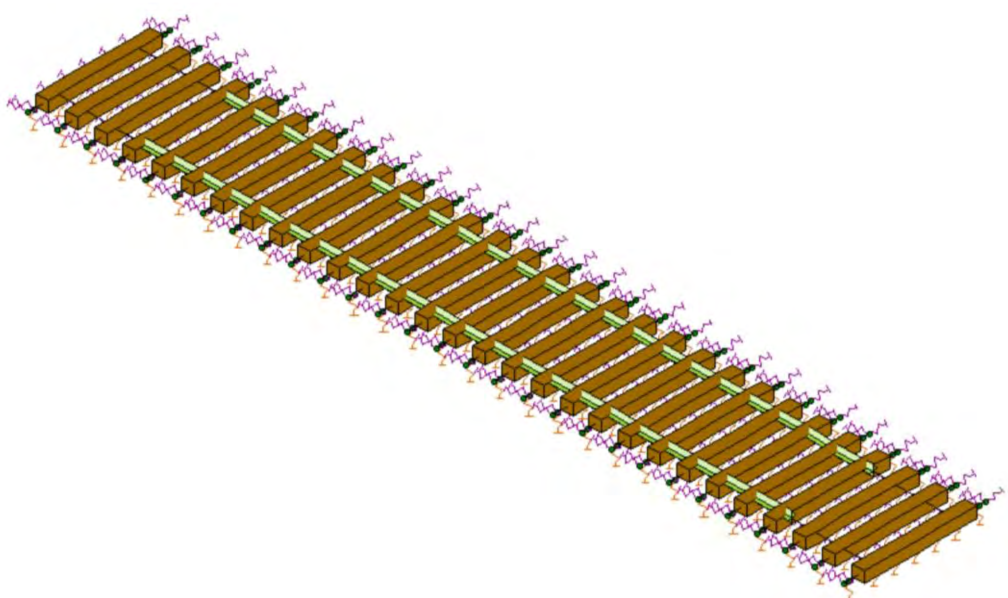


FIGURE 4.3b. Trackbed Model (80% Rendered Length)

4.6 RISA 3D Results

Two 36 kip (160 kN) wheel loads were placed at appropriate nodes above each crosstie. These are simply joint loads incorporated into the model. Figures 4.4a and 4.4b show the results of the trackbed model using a nodal view. Figure 4.4a depicts a side profile view, while Figure 4.4b depicts an isometric view. The results include the self-weight of each individual member. Rail density is set as 490 pcf (7,850 kg/m^3). The nodal view presents corresponding pressure cell-ballast interfacial forces for comparison with the results from Figure 3.8b. These pressure cell-ballast interfacial forces are simply the boundary condition support reactions of the trackbed model.

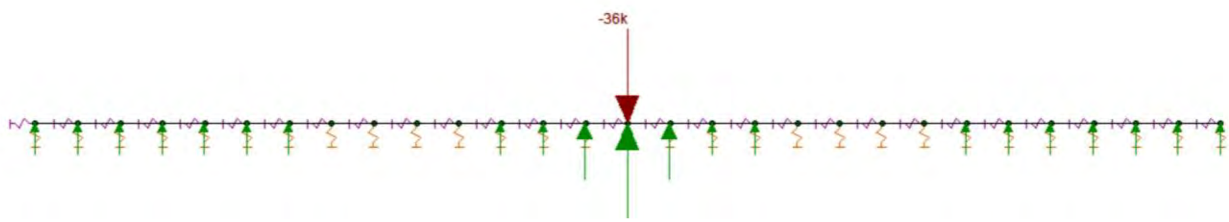


FIGURE 4.4a. RISA Trackbed Analysis Results (Side Profile View)
(Units of Kips) [1 kip = 4.45 kN]

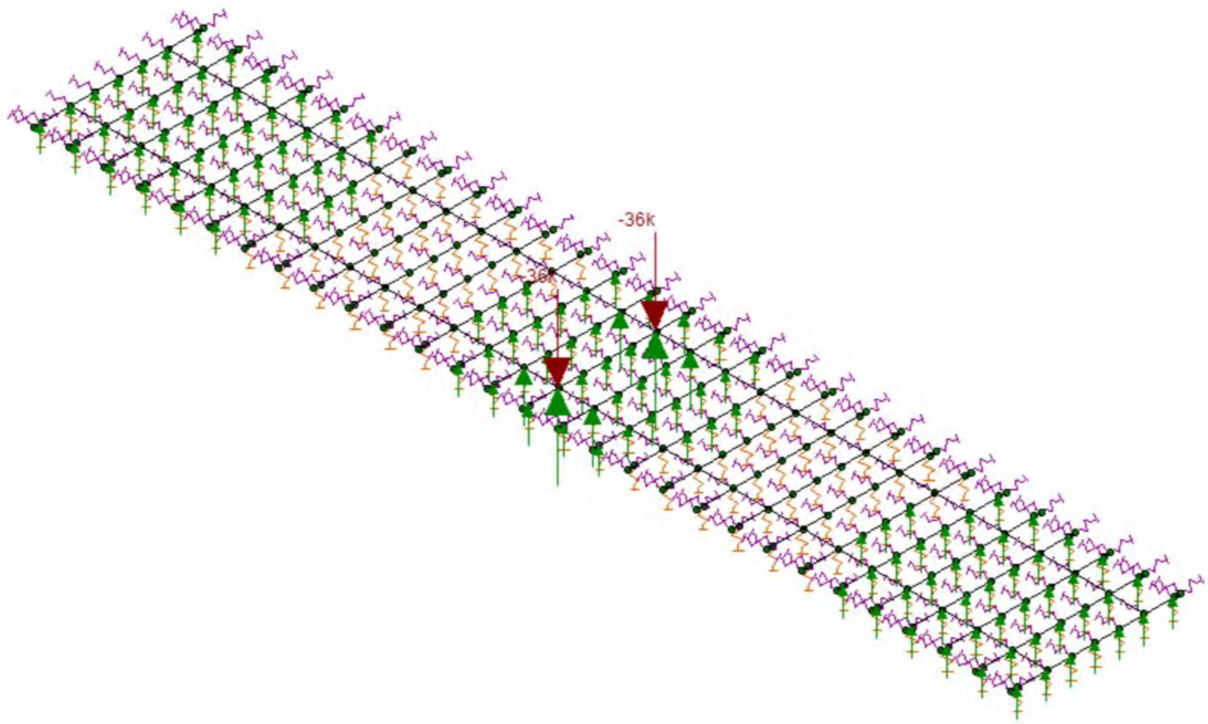


FIGURE 4.4b. RISA Trackbed Analysis Results (Isometric View)
(Units of Kips) [1 kip = 4.45 kN]

Figures 4.4a and 4.4b show support reactions at the ends of the track section. This agrees with the deflected shape and corresponding moment diagram. The crosstie-ballast interface is modeled as compression springs only. This implies that all vertical boundary condition loads shown in these figures are compressive loads.

Figure 4.5 shows a zoomed in view of the critical section of the trackbed corresponding to the area of the model experiencing compressive forces due to direct overhead loading. The load distribution is limited to five crossties as found experimentally, matching Figures 3.8a and 3.8b.

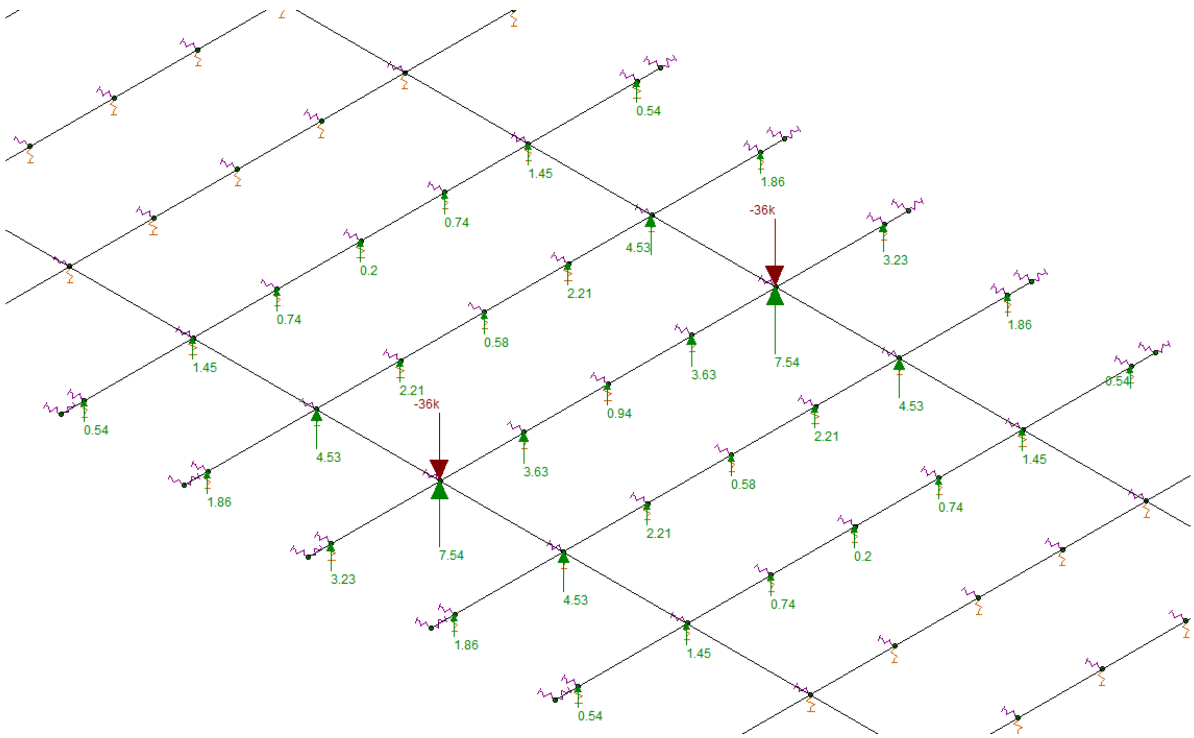


FIGURE 4.5. RISA Trackbed Analysis Results (Zoomed in View)
(Units of Kips) [1 kip = 4.45 kN]

Comparing the results from Figure 4.5 with those expected in Figure 3.8b, there is a strong correlation with some minor discrepancies. The load directly below the crossties and adjacent to the directly loaded crosstie show slightly higher results than those in Figure 3.8b. This matter is discussed in the Conclusion and Recommendations section of this report.

4.7 Poor/Muddy Track

Poor/muddy track support was considered in an effort to view the extent of the load distributing capabilities of the trackbed superstructure. The Kerr procedure was used to calculate a deflection for a Track Support Modulus of 500 lb./in./in. (3,447 kN/m/m) (5). This deflection was used in place of the 1/4 in. (6.35 mm) as used in the main track model. The procedure is outlined below.

Use:

- K = 500 lb./in./in. (3,445 kN/m/m)
- P = 36 kips (160 kN)
- Rail = 140 RE
- Table 1

From the above chosen information, the estimated crosstie deflection for a 500 lb./in./in. (3,447 kN/m/m) Track Support Modulus can be calculated. From Table 1, the w_m/P (in./ton) value corresponding to k = 500 lb./in./in. (3,447 kN/m/m) and 140 RE can be located. This gives:

w_m/P = 0.043311 in./ton (0.124 mm/kN) where, w_m is the deflection and P is the force applied.

Therefore, the deflection is:

Δ = 0.043311 x (36000 lbs./wheel) x (1 ton/2000 lbs.) = 0.7796 in. (19.8 mm)

TABLE 1. Kerr Procedure - Estimated Deflection (5)

k (lb./in./in.)	w_m/P (in./ton)		
	100 RE	119 RE	140 RE
500	0.047553	0.045220	0.043311
750	0.033350	0.031821	0.030567
1000	0.025883	0.024743	0.023814
1250	0.021249	0.020335	0.019597
1500	0.018082	0.017314	0.016700
1750	0.015777	0.015108	0.014581
2000	0.014020	0.013424	0.012960
2250	0.012636	0.012095	0.011679
2500	0.011516	0.011018	0.010640
2750	0.010591	0.010128	0.009779
3000	0.009814	0.009378	0.009054
3250	0.009151	0.008739	0.008435
3500	0.008579	0.008187	0.007900
3750	0.008080	0.007704	0.007432
4000	0.007641	0.007280	0.007020
5000	0.006308	0.005990	0.005767
6000	0.005403	0.005114	0.004915
7000	0.004748	0.004479	0.004296
8000	0.004250	0.003997	0.003827
9000	0.003858	0.003619	0.003458
10000	0.003542	0.003313	0.003159
12000	0.003059	0.002848	0.002707
14000	0.002709	0.002511	0.002379
16000	0.002441	0.002254	0.002130
18000	0.002229	0.002052	0.001934
20000	0.002056	0.001888	0.001775
22000	0.001913	0.001752	0.001644
24000	0.001792	0.001638	0.001534
26000	0.001688	0.001540	0.001439
28000	0.001597	0.001455	0.001358
30000	0.001518	0.001380	0.001287
Note: 500 lb./in./in. = 3,447 kN/m/m			

Using the same method outlined previously in this report, the respective tributary area upscaling factors are applied and give the following values:

K1 = (3.72 kips / 0.7796 in.) / (119 in.^{2} / 459 in.^{2}) = 18.4 kips/in. (3.22 kN/mm)
K2 = (7.89 kips / 0.7796 in.) / (147 in.^{2} / 459 in.^{2}) = 31.6 kips/in. (5.53 kN/mm)
K3 = (3.41 kips / 0.7796 in.) / (128 in.^{2} / 459 in.^{2}) = 15.7 kips/in. (2.75 kN/mm)
K4 = [(0.464 x 2) kips / 0.7796 in.] / (128 in.^{2} / 459 in.^{2}) = 4.27 kips/in. (0.748 kN/mm)

Applying the respective upscale factors grants the following final spring constants rounded to the nearest whole number:

K1 = 129 kips/in. (22.6 kN/mm) K2 = 63 kips/in. (11.0 kN/mm)
K3 = 31 kips/in. (5.43 kN/mm) K4 = 9 kips/in. (1.58 kN/mm)

Figure 4.6 displays the results after replacing the original RISA model's trackbed spring constants with these new poor/muddy track spring constants and running the program.

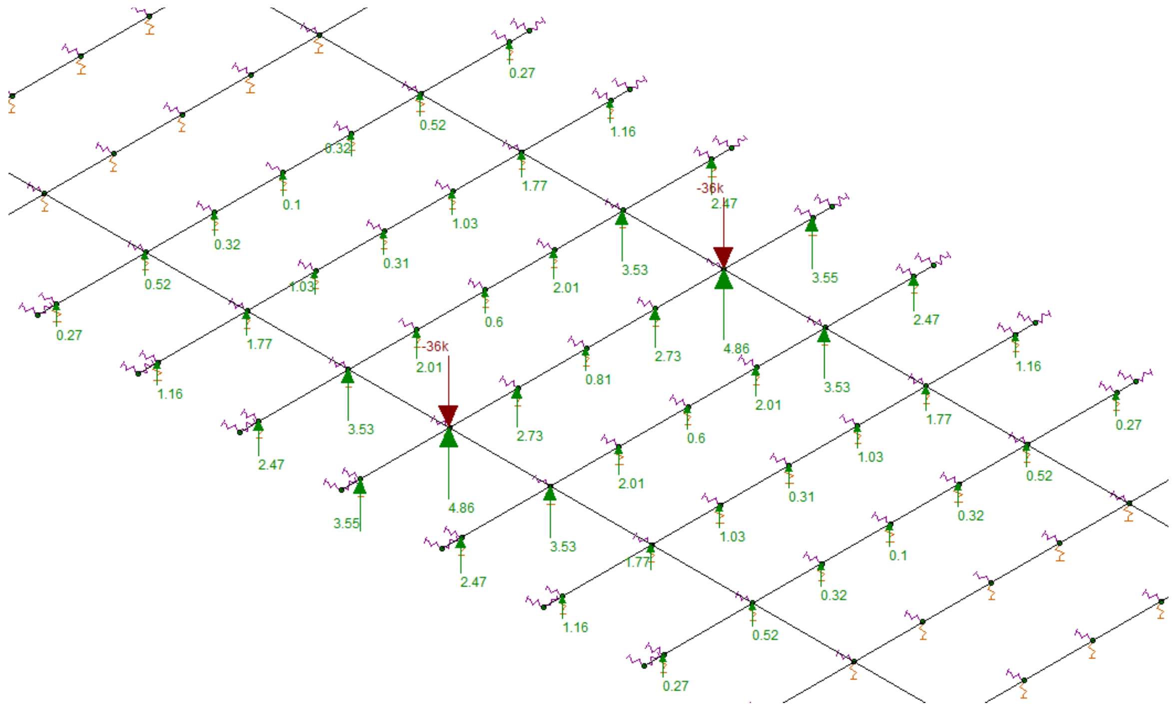


FIGURE 4.6. RISA Trackbed Analysis Results (Zoomed in View) (Poor/Muddy Track Support) (Units of Kips) [1 kip = 4.45 kN]

Notice the extent of the load distribution. The load is distributed across seven crossties rather than five crossties and the proportion of the load carried by any one crosstie has decreased substantially. The extent of the shear felt within the rail in response to the poor/muddy track support can be seen in the RISA Member Forces output file. The rail, through beam action, carries the shear load across the 49 ft. (14.9 m) section. If the model was expanded and depending on the spread of the poor/muddy track support, it seems reasonable to assume that the rail would carry the applied load even farther.

5) SECTION 5. MEASURED VS PREDICTED LOADS

The in-track measured pressure cell load values were graphically compared with the predicted results given by the main RISA trackbed model. Results compared closely with some minor deviation. Figure 5.1a depicts the average lateral load distribution along the directly loaded, full-length crosstie and Figure 5.1b depicts the average longitudinal load distribution between adjacent crossties along the rail.

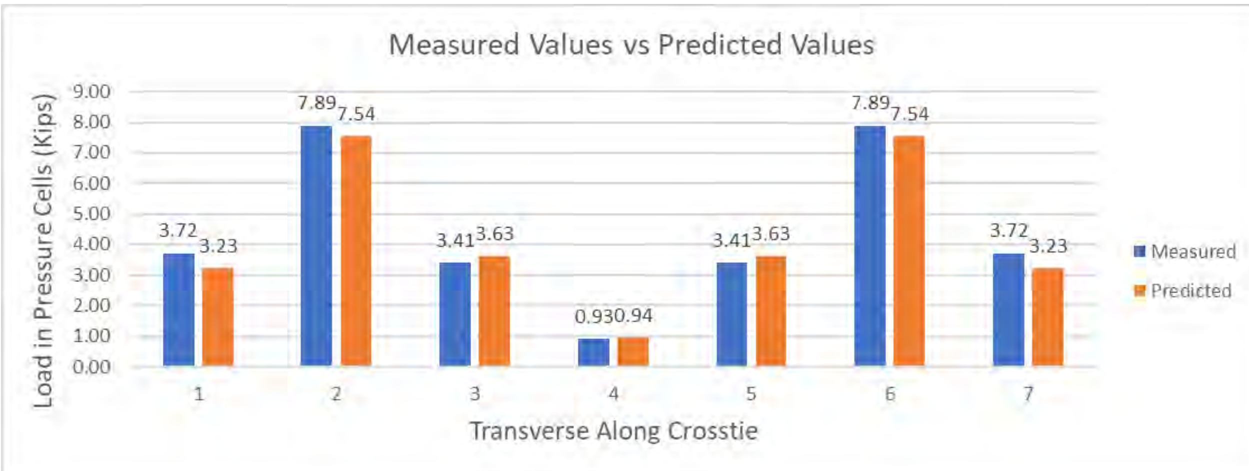


FIGURE 5.1a. In-Track Measured Values vs RISA 3D Predicted Values (Main Trackbed Model) (Units of Kips) [1 kip = 4.45 kN]

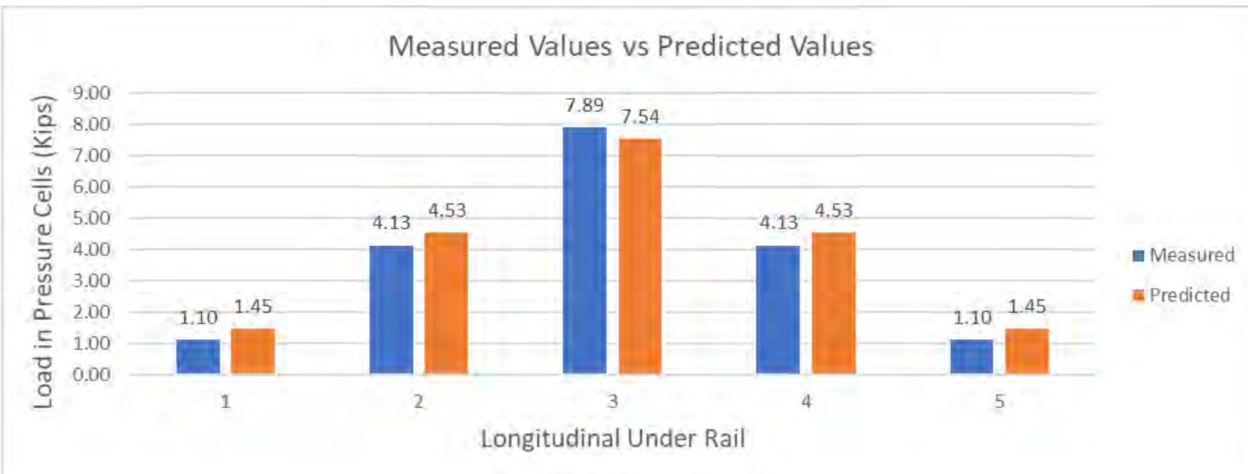


FIGURE 5.1b. In-Track Measured Values vs RISA 3D Predicted Values (Main Trackbed Model) (Units of Kips) [1 kip = 4.45 kN]

6) SECTION 6. SENSITIVITY ANALYSIS

A sensitivity analysis was performed on the original model to determine the extent of the linear range of applicability regarding the chosen spring constants, K. Eight additional RISA runs were conducted using a modified spring constant, K. The spring constants used in the original model were increased by 20% and decreased by 20% in increments of 5%. Poor/muddy track sensitivity analyses were not conducted.

Table 2 contains the results from the sensitivity analysis. The highlighted cells denote the location of the overhead 36 kip (160 kN) wheel load. Table 2 shows the appropriate "Spring Constant, K, Multiplier," the five crossties named 1 through 5 which denote the lateral extent of the load (recall the load is distributed across five crossties), and the node to which the load corresponds to in the table.

The “Spring Constant, K, Multiplier” of 1.00 as seen in Table 2 serves as the baseline for comparison for the conducted sensitivity analysis and are the values contained in Figure 4.5. The variation in the loading results from +20% to -20% is relatively small, but the results appear to be more accurate within the +10% to -10% range. This is likely due to the limits of the linear range used to compute the spring constants via a known target deflection being exceeded past the +10% and -10% margin.

TABLE 2. Pressure Cell Loads using Adjusted Spring Constants, K (Units of Kips) [1 kip = 4.45 kN]

Sensitivity Analysis Results															
Spring Constant, K, Multiplier:	**0.80**					**0.85**					**0.90**				
Tie No.:	**1**	**2**	**3**	**4**	**5**	**1**	**2**	**3**	**4**	**5**	**1**	**2**	**3**	**4**	**5**
Outer Pressure Cell:	0.66	1.98	3.30	1.98	0.66	0.63	1.95	3.28	1.95	0.63	0.60	1.91	3.26	1.91	0.60
Adjacent Pressure Cell:	1.58	4.34	6.92	4.34	1.58	1.55	4.38	7.06	4.38	1.55	1.52	4.44	7.23	4.44	1.52
Adjacent Pressure Cell:	0.83	2.21	3.47	2.21	0.83	0.81	2.22	3.53	2.22	0.81	0.79	2.22	3.57	2.22	0.79
Middle Pressure Cell:	0.23	0.61	0.95	0.61	0.23	0.22	0.60	0.94	0.60	0.22	0.21	0.58	0.93	0.58	0.21
Adjacent Pressure Cell:	0.83	2.21	3.47	2.21	0.83	0.81	2.22	3.53	2.22	0.81	0.79	2.22	3.57	2.22	0.79
Adjacent Pressure Cell:	1.58	4.34	6.92	4.34	1.58	1.55	4.38	7.06	4.38	1.55	1.52	4.44	7.23	4.44	1.52
Outer Pressure Cell:	0.66	1.98	3.30	1.98	0.66	0.63	1.95	3.28	1.95	0.63	0.60	1.91	3.26	1.91	0.60
Spring Constant, K, Multiplier:	**0.95**					**1.00**					**1.05**				
Tie No.:	**1**	**2**	**3**	**4**	**5**	**1**	**2**	**3**	**4**	**5**	**1**	**2**	**3**	**4**	**5**
Outer Pressure Cell:	0.57	1.89	3.25	1.89	0.57	0.54	1.86	3.23	1.86	0.54	0.51	1.83	3.21	1.83	0.51
Adjacent Pressure Cell:	1.48	4.48	7.38	4.48	1.48	1.45	4.53	7.54	4.53	1.45	1.42	4.57	7.70	4.57	1.42
Adjacent Pressure Cell:	0.76	2.22	3.60	2.22	0.76	0.74	2.21	3.63	2.21	0.74	0.71	2.21	3.66	2.21	0.71
Middle Pressure Cell:	0.21	0.59	0.95	0.59	0.21	0.20	0.58	0.94	0.58	0.20	0.19	0.57	0.93	0.57	0.19
Adjacent Pressure Cell:	0.76	2.22	3.60	2.22	0.76	0.74	2.21	3.63	2.21	0.74	0.71	2.21	3.66	2.21	0.71
Adjacent Pressure Cell:	1.48	4.48	7.38	4.48	1.48	1.45	4.53	7.54	4.53	1.45	1.42	4.57	7.70	4.57	1.42
Outer Pressure Cell:	0.57	1.89	3.25	1.89	0.57	0.54	1.86	3.23	1.86	0.54	0.51	1.83	3.21	1.83	0.51
Spring Constant, K, Multiplier:	**1.10**					**1.15**					**1.20**				
Tie No.:	**1**	**2**	**3**	**4**	**5**	**1**	**2**	**3**	**4**	**5**	**1**	**2**	**3**	**4**	**5**
Outer Pressure Cell:	0.49	1.80	3.19	1.80	0.49	0.46	1.78	3.18	1.78	0.46	0.44	1.75	3.16	1.75	0.44
Adjacent Pressure Cell:	1.38	4.61	7.84	4.61	1.38	1.35	4.64	7.99	4.64	1.35	1.31	4.67	8.11	4.67	1.31
Adjacent Pressure Cell:	0.69	2.20	3.68	2.20	0.69	0.66	2.19	3.71	2.19	0.66	0.64	2.19	3.74	2.19	0.64
Middle Pressure Cell:	0.18	0.57	0.95	0.57	0.18	0.17	0.56	0.94	0.56	0.17	0.16	0.55	0.93	0.55	0.16
Adjacent Pressure Cell:	0.69	2.20	3.68	2.20	0.69	0.66	2.19	3.71	2.19	0.66	0.64	2.19	3.74	2.19	0.64
Adjacent Pressure Cell:	1.38	4.61	7.84	4.61	1.38	1.35	4.64	7.99	4.64	1.35	1.31	4.67	8.11	4.67	1.31
Outer Pressure Cell:	0.49	1.80	3.19	1.80	0.49	0.46	1.78	3.18	1.78	0.46	0.44	1.75	3.16	1.75	0.44

Note: Highlighted cells denote location of overhead 36 kip wheel load.

7) SECTION 7. INITIAL TROUBLESHOOTING

The early stages of this endeavor proved troublesome as it was originally attempted to model simply a half-length crosstie as shown in Figures 3.8a and 3.8b with a rail offset to account for its placement on top of the crosstie rather than having its centers of gravity pass through each other as seen in the model depicted in this report. This section of the report briefly discusses these two predicaments.

7.1 Modeling as a Half-Length Crosstie

Because Figures 3.8a and 3.8b are for a half-length crosstie, the initial approach was to create a half-length crosstie model in RISA 3D. This produced many undesired complications, most notably with settling on a value for the boundary conditions. The spring constant originating from the pressure cell in the middle of the crosstie is effectively halved because the other half of the crosstie (or its tributary area) is neglected. In addition, when considering only a half-length crosstie one must also add a one-directional moment restraint at the center of the crosstie to account for the other half of the crosstie’s neglected presence. These difficulties are inherently erased with the modeling of a full-length crosstie. Therefore, the original half-length crosstie approach was abandoned in favor of a full-length crosstie model.

7.2 Rail Offsets

RISA does not have the ability to model a spring outside of a boundary condition; therefore, the initial models included an arbitrary member with adjusted Modulus of Elasticity and area values to act as a rail-crosstie spring to potentially better influence the distribution of load. It is known from prior studies that the rail will deflect around 0.06 in. (1.52 mm) upon loading (8), but this is in the middle of the rail's unsupported length. So, the question is how to take this deflection and back calculate an adjacent crosstie deflection. This is difficult as one cannot model the track as simply supported or as a multi-span, simply supported beam with a set number of spans.

It may be possible to estimate the adjacent crosstie deflection by isolating that section of track and modeling it as a multi-span, simply supported beam with an effective length or span stretching from the point of loading to either side of the applied load. However, given that the initial deflection is only a miniscule fraction of an inch, it was deemed negligible. It seemed logical that if the arbitrary members were not going to be designed for a known deflection, then why include the member in the model at all. Therefore, it was decided to treat the track support as the only springs in the model.

8) SECTION 8. CONCLUSION AND RECOMMENDATIONS

This report outlines the procedure used to perform a finite element analysis on a timber crosstie trackbed superstructure using RISA 3D to recreate and predict the load distribution at the crosstie-ballast interface as seen in measured field data. Based on prior research performed at the University of Kentucky, close estimates obtained for the crosstie-ballast load distribution are provided in Figures 3.8a and 3.8b. From this data, a finite element model displayed the load distribution. It is unclear whether any significant degrees of structural modeling for analysis purposes are used for trackbed design and evaluation. Nor is it clear that anyone truly grasps the extent of the load carrying and distributing capabilities of a railroad trackbed. The modeling approach contained herein could potentially be used in trackbed designs or analyses.

The modeling method described herein is idealized for a fully tamped, ballast trackbed support and is inherently limited to the initially assumed Track Modulus (conventionally 3000 lb./in./in. (20,684 kN/m/m) for tamped, wood crosstie track) (assumed 500 lb./in./in. (3,447 kN/m/m) value for muddy track). The selected Track Modulus value is used to compute the approximate linear spring constants for use in the model.

In calculation of the linear spring constants, the methodology outlined herein assumes an even (or average) total track deflection value along any single crosstie with no deflection variation along the crosstie (i.e., each full-length crosstie when subjected to 36 kip (160 kN) overhead wheel loads deflects 1/4 in. (6.35 mm)). In actuality, there are likely different deflection variations across any one loaded crosstie, and perhaps could be modeled more appropriately using non-linear springs.

The underlying data that was used to create Figures 3.8a and 3.8b reference just one set of data. The results from the RISA model contained herein and shown in Figures 4.4a through 4.5, vary slightly if data from other data sources are used. However, the scale of difference is small, with most of the variability arising at the pressure cells directly under the rail. The reference data used in this report provides a reasonable estimate for the extent of the load distribution.

In-track measured pressure cell values compared with the predicted results given by the main RISA trackbed model showed great similarity. Deviation between respective values was minor, providing added confidence in the RISA model itself.

Poor/muddy track support conditions used the same modeling approach as used in the model depicted in Figures 4.4a through 4.5. It would be interesting to compare these results with actual poor/muddy track data. Figures 3.8a and 3.8b could be recreated using this poor/muddy track support data and compared to either the results from this report's modeling approach or another finite element modeling approach.

The spring constants used in this report would be specific to the layout or location of the pressure cells in the timber crossties as detailed in Figure 4.1. It would be interesting to determine how much variability these spring constants would experience if the location of the center of the pressure cells, and thus the location of the compression springs, were changed.

It seems evident from the consistent use of target deflections to obtain values for the subgrade spring constants that a railroad trackbed is not linear as modeled herein, but is likely nonlinear. The approach detailed in this report provides an adequate working model to estimate the load distribution at the crosstie-ballast interface, but in future renditions of this research, perhaps using a program capable of utilizing nonlinear springs, such as SAP2000, could provide improved and more accurate results. In addition, there has been some advancements in discrete element modeling of railroad trackbeds. This program is advantageous if the engineer would like to consider the effects of each individual trackbed support layer separately (i.e., ballast, subballast, subgrade, etc.), rather than as a whole in the case of the finite element model depicted in this report.

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10) SECTION 9. AUTHOR CONTRIBUTION STATEMENT

The authors confirm contribution to the report as follows: study conception and design: B. Thompson; data collection: B. Thompson, J. Rose; analysis and interpretation of results: B. Thompson; draft manuscript preparation: B. Thompson, J. Rose, D. Clarke. All authors reviewed the results and approved the final version of the manuscript.

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12) SECTION 10. ACKNOWLEDGEMENTS

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